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JUNE 1959
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OF THE
AMERICAN CONCRETE INSTITUTE

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"Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)" supersedes the 1942 standard. It was prepared by ACI COMMITTEE 614.

●

PHIL M. FERGUSON and FARID N. MATLOOB present their findings on the "Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Beams." Cutting off reinforcing bars in tension zones is shown to lower beam shear strength. Both bond stress and diagonal tension act to reduce strengths.

"Prestressed Overlay Slab for San Antonio Airport," coauthored by M. M. LEMCOE and C. H. MAHLA, discusses design criteria, theoretical calculations, construction procedures, stress measurements, and performance history to date.

J. O. HENRIE describes general shielding requirements, and concrete properties and mixes which best meet these requirements in his paper "Properties of Shielding Concrete."

●

"Width of Cracks in Concrete at the Surface of Reinforcing Steel Evaluated by Means of Tensile Bond Specimens," prepared by D. WATSTEIN and R. G. MATHEY, describes performance of a tensile bond specimen which simulates a segment of the tensile portion of a reinforced concrete beam between two successive tensile cracks.

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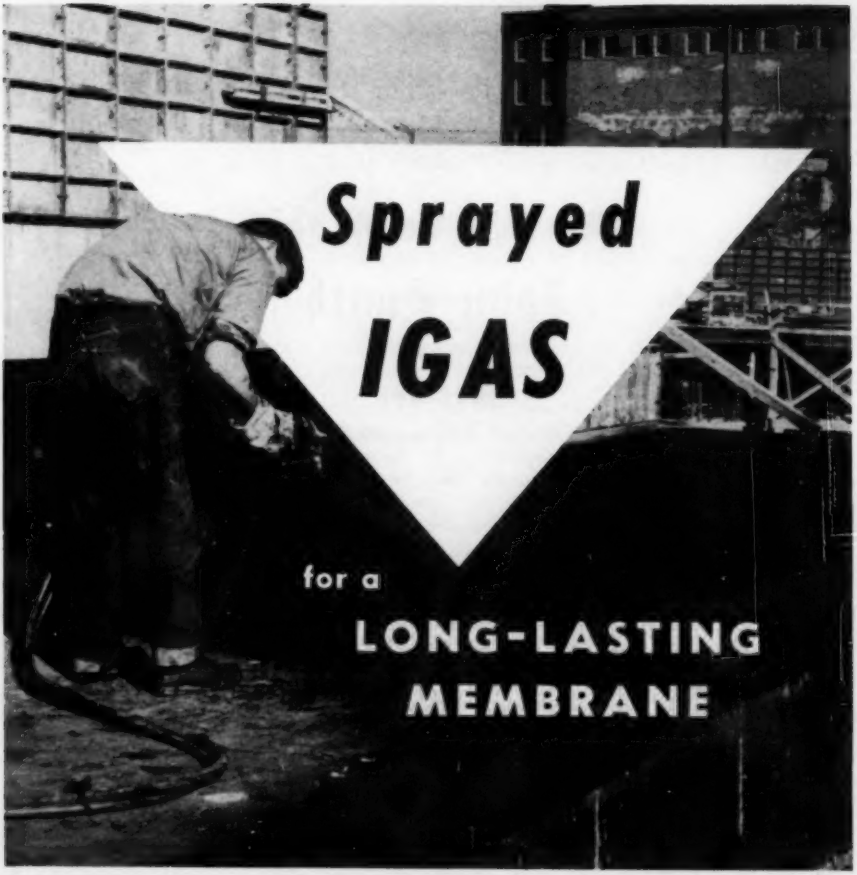

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Title No. 55-75

Use of High-Strength Steel in Reinforced Concrete

By GEORG WÄSTLUND

In Europe the use of high-strength steel for reinforcing bars is increasing and is, to a degree, replacing reinforcement made of mild steel. Both Austria and Sweden have developed reinforcing steel with high yield points (50,000 psi or more), one employing cold working and the other employing "natural" elements, i.e., more carbon and better alloys. This steel increases working stresses and, generally, eliminates the need for end hooks. Failure precautions, crack formation, and deflection considerations limit the full use of this steel.

THE TERM "HIGH-STRENGTH STEEL," as used in what follows, denotes reinforcing bars having yield points in excess of 50,000 psi. Furthermore, high-strength steels must have a considerably higher bond resistance and/or a higher anchorage resistance than ordinary mild steel bars.

High-strength steels came into use in the early 1940's in Sweden and in Austria. The trend of development since that time has been toward progressively higher steel qualities and higher permissible stresses. At the same time, the quantity of high-strength steels employed for concrete reinforcement has substantially increased in relation to that of mild steels. From the beginning, the development in Sweden and in Austria has proceeded on essentially different lines. The Swedish high-strength steel, Kam steel, is a "naturally hard" steel. That is, its high strength is due, first, to a comparatively high carbon content, and second, to certain alloying elements. On the other hand, the high strength of the Austrian Tor steel is obtained by cold working.

During the past few years, in addition to these two types of steel, several new variant grades of high-strength steels have been placed on the market. As a result, different types of reinforcing steels are now available in such great variety that it may seem to be too diversified for requirements of rational manufacture and stock-keeping in some countries.

Georg Wästlund is professor of bridge building and structural engineering, and director of the Swedish Cement and Concrete Research Institute, Royal Institute of Technology, Stockholm. Professor Wästlund worked as a laboratory assistant in the Structural Strength Laboratory of the Royal Institute and as a structural engineer at Skanska Cementgjuteriet before assuming his present position at the Royal Institute in 1941. He has been a member of ACI since 1946 and is presently a member of ACI Committee 339, Allowable Stresses in Reinforcement. Professor Wästlund received his BS in 1928 and his doctor's degree in 1934.

Fig. 1 shows the Swedish Kam steel 40, which has transverse deformations as well as two longitudinal ribs. The cross-sectional area of the transverse deformations and the spacing of these deformations on bars of this type agree closely with American deformed bars, which have in part served as a model for the design of Kam 40. Thus, the spacing of the deformations is equal to about $0.80d$ (as against $0.70d$ in the United States), and the "specific cross-sectional area" of the deformations, i.e., the projected cross-sectional area of a deformation at right angles to the bar divided by the spacing of the deformations, is equal to $0.20d$ (i.e., about the same as in the United States). On the other hand, the yield point of Kam 40 is slightly higher. Thus, the requisite yield point ranges from 51,000 to 57,000 psi according to the diameter, while the American hard grade deformed bars have a yield point of at least 50,000 psi. The permissible tensile stresses stipulated in the Swedish standard specifications are substantially higher. They vary from 25,000 to 31,000 psi, whereas the corresponding American figure is 20,000 psi.

At the present time, two other standard types of Kam steel, viz., Kam 50 and Kam 60, are also used in Sweden. Kam 60 (Fig. 1) has a smaller spacing of transverse deformations than Kam 40, about $0.50d$, a specific cross-sectional area of the deformations that is twice as great, i.e., $0.40d$, and a requisite yield point of 85,000 psi.

The allowable stresses for Kam 50 and Kam 60 vary from 34,000 to 42,000 psi. Subject to special permission, they may even be increased to 47,000 psi.

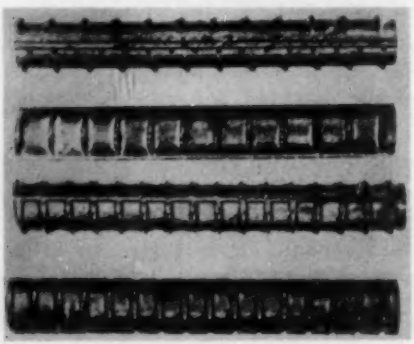

Fig. 1—Swedish Kam steel 40 (above), with yield point from 51,000 to 57,000 psi, and Kam steel 60, with yield point 85,000 psi

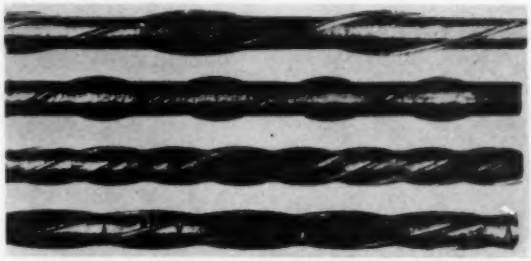

Fig. 2—Various types of Tor steel 60

On the other hand, the maximum diameter of Kam 60 is limited to $\frac{5}{8}$ in., and this is an important point.

The Austrian Tor steel is provided with helical ribs, as shown in Fig. 2. It is available in three grades, Tor 40, Tor 50, and Tor 60. Their proof stresses are roughly equal to the yield points of Kam 40, Kam 50, and Kam 60, respectively. Tor 40 is made of ordinary mild steel, which is rolled into round bars with longitudinal ribs. After rolling, the bars are subjected to torsion in a special cold-twisting process, from which the name "Tor," an abbreviation of torsion, is derived. This cold-working process increases the yield point as well as the ultimate strength, whereas it reduces the elongation.

As Tor steel is cold worked, its stress-strain diagram does not exhibit any marked yield point, as is seen from Fig. 3.

The effect of cold working on the strength properties of the steel is represented in principle in Fig. 4. The increase in the ultimate strength is influenced by the phenomenon of aging. This phenomenon is a slow change in the properties of steel during the first few months after cold working.

In Sweden Kam steel has largely replaced plain reinforcing bars made of mild steel. This is primarily due to the price, which favors Kam steel in that the additional expense is relatively smaller than the increase in the

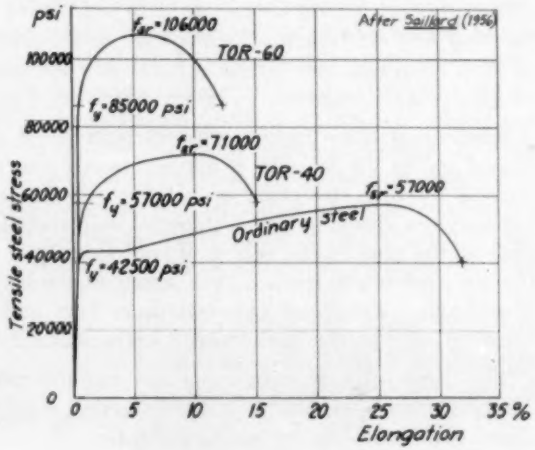

Fig. 3—Stress-strain diagrams for ordinary steel, Tor steel 40, and Tor steel 60. (After Saillard, 1956)

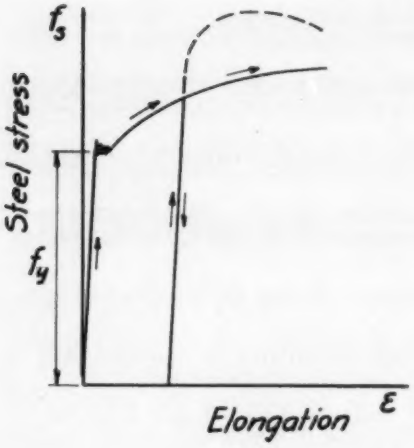

Fig. 4—Stress-strain relations as influenced by cold working and aging

allowable stresses. In the second place, this trend has been furthered by the circumstance that end hooks can be eliminated. This offers two advantages, first, a working operation is eliminated, and second, a saving in materials is possible.

FURTHER DEVELOPMENTS

As has already been pointed out, many other types of high-strength reinforcing steels have been introduced in recent years. It may be of interest to mention at least two of these types. Both of them are cold-worked.

One of those comparatively recent types of reinforcing steels is Tentor steel, which is a Danish innovation (Fig. 5). It is characterized by oblique ribs along both halves of the bar. Its yield point is increased by simultaneous twisting and stretching. Tentor bars can therefore be made of lower grades of steel than required for Kam bars. On the other hand, the manufacturing process is more intricate. The yield point of Tentor bars is 71,000 psi.

Another of these recent types of reinforcing steels which should be mentioned is "Ss 70 A." It is a plain round bar whose 0.2-limit has been increased by cold stretching to a value that is as high as 100,000 psi. It is furthermore characterized by special end anchors, which are wedged on the ends of the bars in the course of cold stretching (Fig. 6). This type of reinforcing steel was developed by Forssell, at the Royal Institute of Technology, Stockholm. The bond between these bars and the concrete is not particularly good, but the anchorage is extraordinarily reliable.

This type of reinforcing steel has recently been standardized in Sweden. The permissible stresses are under certain conditions as high as 57,000 psi. These conditions will be mentioned later.

REQUIREMENTS FOR STRUCTURES

All engineering requirements which a reinforced concrete structure has to fulfill may roughly be classified under the following three main headings:

1. Adequate safety against failure
2. Limited crack formation
3. Limited deflection, i.e., sufficient rigidity

The European Concrete Committee CEB has appointed several working groups to deal with special problems within this scope.

The committee has made the following general statement, among others: simplified calculations of the load-carrying capacity of the structure shall be made irrespective of the check calculations relating to crack formation and to deflection. The latter check calculations shall refer to the stage of use, and shall be carried out separately.

This general statement has been followed by Swedish authorities, and has been embodied in detailed specifications concerning the Ss 70 A reinforcing steel.

Thus, it is quite likely that the high permissible steel stresses referred to above can be seldom utilized, because the requirements relating to limitation of crack formation may be wholly decisive in some cases, while the requirements regarding the limitation of deflections may be decisive in others. Accordingly, we have, so to speak, several "ceilings" involving quite different parameters which are applicable at the same time. Which of these ceilings will be the deciding factor is a question that depends on circumstances.

Requirements for adequate safety against failure

A structure can fail in many ways. As a general rule, high-strength steel reinforcement, which renders possible higher permissible stresses, shall ensure adequate safety against failure in several respects. We can distinguish between the cases enumerated in what follows.

Failure in bending or in combined bending and compression—The extensive studies of ultimate load design which have been made in the United States and Europe have to a large extent elucidated the factors or parameters which determine the ultimate load in bending. Fairly reliable calculations of the ultimate load in pure bending, in composite bending and compression, and in pure compression can be made for various grades of reinforcing steels and for several types of concrete differing in strength. However, this statement is made with the reservation that the reliability of these methods of design in the cases where the failure of the concrete in compression is decisive is substantially smaller than in the cases where the yield of the rein-

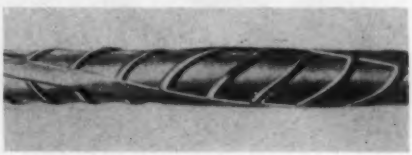

Fig. 5—Danish Tentor bar, yield point 71,000 psi

forcement is the deciding factor. Thus, the fact that the relation between the ultimate load and the reinforcement is fairly well known makes it possible to increase the permissible stresses approximately in proportion to the yield point.

There is another reservation to be made. If the method used for calculating the moments within the structure is unreliable, and if this method gives values which *may* be on the unsafe side, then it is obvious that special caution is needed in determining the permissible stresses. For instance, this will for the time being probably be the case where use is made of the simple moment coefficients for flat slabs specified in the ACI Building Code. Therefore, when the ACI coefficients are used in Sweden, and when unfavorable cases of loadings can occur, care is taken that the permissible stresses do not exceed the values given in the ACI Code.

Failure in shear—What has been said above, about the necessity of caution in the cases where the theory of calculation of ultimate loads is unreliable, can in particular be applied to failure in shear. In this case, the uncertainty is much greater than in the calculation of the ultimate load in bending. The uncertainty and the shortcomings of the classical methods of design are unfortunately evident enough even in the cases where ordinary reinforcement is employed. This has perhaps been realized in the United States earlier than in other countries. Accordingly, extensive investigations have been initiated to study these problems.

It may be said that the problem under consideration is still further accentuated when use is made of high-strength reinforcing steel of the Kam bar type. This circumstance has been emphasized, although from different points of view, by Hajnal-Kónyi, Hillerborg, and Granholm. For instance, as has been demonstrated by Granholm, after the appearance of the first shear crack in a beam reinforced with ordinary plain round bars, which are provided with end hooks, but not equipped with stirrups, the load can further be increased considerably, sometimes to twice the original value, before definite failure will take place. In other words, failure is forewarned.

This is not always the case when a corresponding beam is reinforced with Kam steel bars without end hooks. Here, failure can occur immediately after the appearance of the first shear crack, suddenly, and without warning.

After the appearance of a shear crack, such a beam no longer acts in conformity with the classical theory of beams. This phenomenon can be expressed according to Granholm, by stating that the beam is more or less

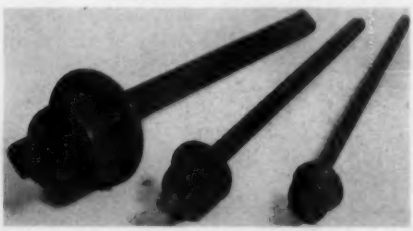

Fig. 6—Swedish Ss 70 A, cold worked plain bars with special end anchors. Yield point 100,000 psi

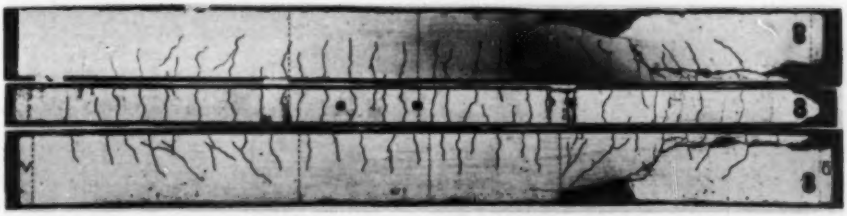

Fig. 7—Three views of typical crack formation and rupture of a beam with Kam steel and no stirrups. (After Granholm, 1958)

converted into something similar to a bowstring girder. Or it can also be said, according to Hillerborg, that the moment point for the determination of the tensile force in the reinforcement is moved toward the center of the beam, with the result that the tensile force is increased.

In the presence of end hooks, the increase in the tensile force can as a rule be taken by these hooks. In the absence of end hooks, on the other hand, the bond stresses increase, and might cause a sudden bond failure (Fig. 7).

Hajnal-Kónyi has pointed out that it is "essential to provide stirrups in sufficient quantity so that the tensile stresses in the concrete can be taken up by them should the tensile strength of the concrete be suddenly exhausted."

Slip and failure in anchorage—The Kam steel bars in Sweden, just as the deformed bars in this country, are anchored by bond, and are not provided with hooks. The allowable bond stresses for Kam 60 and Kam 40 are equal, in spite of the fact that the specific cross-sectional area of the transverse ribs of the former is twice as great as that of the latter. This might seem surprising, but it is to be regarded as a precautionary measure.

Longitudinal splitting due to irregularities on the surface of the reinforcing bars—The risk of longitudinal splitting due to the transverse ribs of the Kam bars can be counteracted in two ways, viz., by ensuring a sufficient concrete cover and by providing stirrups.

Local splitting of the concrete at bends of the reinforcing bars—The risk of local splitting or local crushing due to high local stresses in the concrete at bends of reinforcing bars is to be regarded as serious. If failure is due to this cause, then it occurs without warning, and sends concrete fragments flying in the air like shot.

Unfortunately, only a few tests have been made with a view to clarifying these phenomena, and the tests in question are now quite old. The author carried out some small tests of this kind about 20 years ago. Fig. 8 shows the type of failure, characterized by splitting. The results obtained from these tests are reproduced in Fig. 9. As is seen from these results, the ultimate load increases as the radius of curvature of the bend increases, but is by no means proportional.

At a radius of curvature equal to $6.2d$, failure still was due to splitting of the concrete. At larger radii the steel yielded, but the steel was only mild steel.

Fig. 8—Splitting of concrete at a bar bend in tension. This type failure is local and occurs without warning. (Wästlund, 1934)

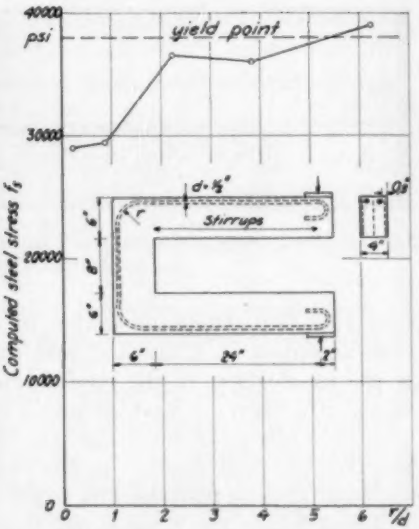

Fig. 9—Tests on splitting caused by bar bends in tension. Concrete cube strength 3060 psi, i.e., cylinder strength ≅ 2620 psi. (Wästlund, 1934)

Other tests which the author has made on reinforcing loops demonstrated that the splitting strength did not increase in proportion to the strength of concrete either, but was approximately proportional to $(f_c')^{3/4}$.

Tests carried out at about the same time by Graf gave more or less similar results, although he found that the splitting strength increased at a still lower rate in relation to the strength of concrete. Graf used a higher grade of reinforcement, and was therefore able to increase the radii of curvature further than the author. For $f_c' = 5300$ psi and $r = 10d$, he observed splitting failure at a steel stress of 54,000 psi.

In standard specifications and in design, the phenomena under consideration can be taken into account in two alternative ways. One way is to increase the radius of curvature as the stress in the reinforcement becomes greater. This has been stipulated, for instance, in the Austrian regulations. The other way is to keep a constant and relatively large radius of curvature, and then to vary the permissible steel stress in accordance with the strength of concrete. This has been done to a certain extent in the Swedish standard specifications. Neither of these methods seems to be based on sufficient experimental evidence. Further tests are therefore highly desirable.

It should be added, however, that it is considered advisable in Sweden, for the time being, to bend Kam 60 bars to a limited extent only. Besides, these bars are difficult to bend because of their high strength.

Requirements for limitation of crack formation

It has already been mentioned that designs should also be checked with respect to crack development. The question of limitation of crack formation is in itself a separate subject of wide scope.

To begin with, emphasis should be given to the pioneer work on crack formation problems by Watstein and Parsons (1943).

It is obvious that the danger of development of large cracks in structures reinforced with high-strength steels, which render possible high permissible stresses, is greater than it is in structures which are subjected to low stresses.

However, extensive investigations have shown that the parameters which determine the crack widths also contain essential quantities other than the steel stress. Consequently, it is possible to exert an advantageous influence on crack formation within relatively wide limits, and hence to compensate the unfavorable effect of high steel stresses.

The crack problem is most commonly studied in two stages,

1. Determination of empirical rules for estimating crack *spacing*
2. Determination of the relationship between crack spacing and crack *width*

Several crack formulas have been derived. However, the value of crack formulas is limited. There are so many factors involved in this problem that it is quite impossible to consider them all in practice. Thus, for instance, weather conditions and restrained or free shrinkage are of considerable importance in practice, and their effects are partly uncontrollable. Furthermore, the problem in itself is of statistical character, that is to say, the occurrence of cracks is to a large extent accidental.

The main purpose of a crack formula is to serve as a guide in ensuring that the structure shall be sound with respect to crack formation. Several existing crack formulas fulfill this purpose satisfactorily.

Most formulas for estimating crack spacing are of the following type:

$$a_m = C_1 + C_2 \frac{f_r A_c}{u_m \sum o}$$

where

a_m = mean crack spacing
f_r = concrete tensile strength
u_m = bond strength
A_c = cross-sectional area of the tensile zone of the beam or a revised cross-sectional area of this zone
$\sum o$ = sum of the bar perimeters
C_1 and C_2 = coefficients

Then the *mean crack width* w_m is found from a formula which, in its simplest form, can be written

$$w_m = a_m \frac{f_s}{E_s}$$

where

f_s = steel tensile stress in cracked section
E_s = steel modulus of elasticity

It is immediately seen that a high steel stress f_s can be compensated by a small value of crack spacing a_m.

This can be achieved in several ways,

1. By using bars having a high bond strength u_m
2. By using bars of small diameter, which results in a high value of $\sum o$
3. By keeping the cross-sectional area of the tensile zone of the beam as small as possible with due regard to other requirements, such as the necessary concrete cover
4. By ensuring a uniform distribution of the bars over the tensile zone of the beam. This is particularly important in the flange of a T-beam, which is subjected to a negative moment

The possibilities of influencing the crack formation in these ways are great, but here will be confined to one example only. Compare Fig. 10 and 11 showing two T-beams having the same amount of reinforcement but different bar diameter and different arrangement of reinforcement.

The same conclusions as above can be drawn from a formula that is quite different from those which have just been shown. This formula was deduced by Jonsson, Osterman, and the author (1945), and has been used in Sweden since 1947.

Fig. 10—Crack formation in a T-beam, subjected to negative moment, reinforced with few, rather thick, plain round bars, standard steel St 52, concentrated to the middle of the flange. A few large cracks, crack widths up to 0.046 in. at tensile steel stress only 22,700 psi. (Wästlund and Jonsson, 1948)

Fig. 11—Crack formation in a T-beam, subjected to negative moment, reinforced with many small plain round bars, standard steel St 52, well distributed over the flange. Many small cracks, crack widths up to only 0,0046 in. at the same tensile steel stress as the beam shown in Fig. 10. (Wästlund and Jonsson, 1948)

For the case of *pure bending* this formula is as follows.

$$w_{max} = kD\left(\frac{I_c}{dA_s e_t}\frac{f_s}{E_s}\right)^{2/3}$$

where

w_{max} = calculated maximal crack width, in.
k = 0.23 for plain bars
k = 0.16 for deformed bars (Kam steel 40)
D = diameter of bar, in.
d = depth from compression face of beam to centroid of longitudinal tensile reinforcement, in.
A_s = cross-sectional area of reinforcement, sq. in.
f_s = working steel stress, psi
E_s = modulus of elasticity of steel, psi
I_c = moment of inertia of the full concrete section about the neutral axis, in.;
e_t = distance from neutral axis to extreme fiber on tension face, in.[4]

In fact, the calculation of crack widths according to this formula has been compulsory in Sweden for all highway bridges since 1947. The maximum allowable crack width calculated from the crack formula corresponds to values of about 0.012 in. for the dead load alone, and about 0.016 in. for the sum of the dead load and half the live load.

In 1958, new tentative standard specifications for the use of Ss 70 A steel were published in Sweden, as already mentioned. In view of the high permissible tensile stresses, these specifications also include definite restrictions relating to calculated crack widths. Thus, the calculation of crack widths is compulsory for all uses of this particular type of bar, i.e., not only for highway bridges, but also for other structures. For other structures, however, under those conditions which are not liable to give rise to corrosion, the calculated crack widths are allowed to be somewhat larger.

As said before, the check of designs with respect to cracking will often lead to the result that the maximum allowable stress cannot be used at all. This means that the restriction of crack widths is frequently more severe than the upper limit for the permissible tensile stress.

Requirements for limitation of deflection

When use is made of high-strength reinforcing steels, which involve high stresses, the deflections are also often regarded as a decisive factor. This implies that the maximum permissible tensile stresses cannot be utilized. This circumstance deserves close attention.

It cannot be said, however, that any generally applicable method accepted in standard specifications has been evolved for taking into account the crack formation, the shrinkage of concrete, and the plastic deformation of concrete under the action of sustained loads.

Certain scattered test results are available in this connection. They are of great interest, but they also show that the problem under consideration is not so simple as it might have appeared at the outset.

Furthermore, it is to be noted that the requirements for limitation of deflections must be varied from one case to another. In house construction, for instance, the concrete floor slabs which have to carry masonry partitions must comply with severe requirements in this respect, because deflection of a floor slab causes the development of ugly cracks in the partitions. For this special case, however, the new Swedish standard specifications contain provisions concerning the calculation and the limitation of deflections.

BENDING PROPERTIES OF REINFORCING BARS

Ductility can be measured either in terms of the ultimate elongation, which is conventionally determined in a length of $10d$, including the fracture section, or in terms of the uniform elongation which is defined as the elongation attained just before contraction sets in, i.e., when the stress in the bar has just reached its maximum value. Both these elongation values seem to be useful, but the former is mainly employed for naturally hard steels, whereas the latter often is used for cold-worked steels.

In addition to the elongation test, it also appears necessary to make a bend test, which is indeed usually the practice.

ASTM specifications prescribe that hard grade deformed bars "shall stand being bent, at room temperatures, around a pin without cracking on the outside of the bent portion," the diameter of the pin being equal to six times the diameter of the bar, and the angle of bending being 90 deg. The corresponding Swedish standard specifications are slightly more severe, the respective values being $5d$ and 180 deg (Fig. 12).

The German standard specifications are much more rigorous in this respect. They require first a usual bend test, 180 deg, around a pin $2d$ to $4d$ in diameter, then a new bend test, 45 deg, around a pin $4d$ to $8d$ in diameter,

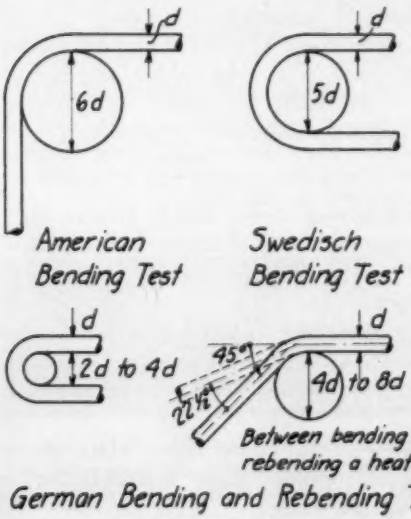

Fig. 12—Comparison between American, Swedish, and German bending tests

and after that rebending 22½ deg (Fig. 12). In fact, these requirements seem suitable for indicating the possibility of brittle fracture.

However, requirements which are as severe as that cannot be fulfilled without entailing additional expenses. They must raise the price of steel.

On the other hand, it should be fully realized, that the Swedish regulations, and hence also the American requirements in this respect, are in many cases far from being satisfactory. In practice it is often necessary to bend a bar sharply and then also to rebend it. For instance, this is the case when use is made of sliding forms and connection bars for a slab must temporarily be bent sideways.

The solution to be adopted in such cases seems to be to prohibit the use of standard high-strength steels for the purpose in question, and to employ special steels having particularly good bending and rebending properties.

It should also be noted that high-strength steels are usually sensitive to low temperatures, and tend to become brittle at these temperatures. Thus, rough handling alone may be sufficient to cause fracture of these steels at low temperatures.

Another risk of insufficient ductility is that microcracks may occur in the bars on account of deformations due to bending. These cracks can appear on the *inside* of the bent portion, quite contrary to the above-cited ASTM specifications. It is true that no accidents due to such cracks have been reported as yet, but further investigations on this point seem desirable.

As a matter of fact, Kam 60 bars are not easy to bend, or to bend correctly. Therefore, these bars are most often used without bends.

CLOSING REMARKS

American practice has proved the possibility of using deformed bars, and has demonstrated their advantages, i.e., dispensing with end hooks, ensuring better bond, and reducing the size of cracks.

European practice has benefited by American practice in this field, and has advanced a little farther by using steels of higher strength, which enable higher permissible stresses, and thus result in more economical designs.

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OVER 400 REINFORCED MORTAR PRISMS mixed with water from the Mediterranean Sea were tested for corrosion of reinforcement at ages ranging from 3 months to 4 years. Variables studied were water-cement ratio, cement content, and storage conditions. The pH necessary for inhibition of corrosion by hydroxyl ions under different storage conditions was determined, and pH values of mortars and cement pastes made with sea water and fresh water were measured.

The reinforcement was corroded in all specimens stored in moist air, but for mortar prisms stored in sea water or tap water practically no corrosion of reinforcement was observed. No regular relationship between consistency or cement content on the one hand and amounts of rust on the other was found. Corrosion did not halt but, with one exception, was still increasing as the study ended. Conclusion is drawn that sea water used for mixing mortar or concrete for air-exposed reinforced structures tends to make reinforcement highly vulnerable to corrosion.

Influence of sea water on corrosion of reinforcement

R. SHALON and M. RAPHAEL

SEA WATER IS SOMETIMES USED as mixing water for concrete, and it often penetrates concrete either through direct contact or in the form of spray. Opinions differ on the question of sea water as a cause of corrosion of reinforcement. Most data available on the effect of sea water bear on its effect on strength. In fact, the influence of sea water on the rate of corrosion of reinforcement has not been studied systematically, and available data are limited, far from clear, and sometimes contradictory.

As a result of this lack of clarity certain specifications and codes permit the use of sea water for mixing concrete, while others either restrict or prohibit its use, or make no reference to its effect. The German specifications[1] permit the use of all kinds of water occurring in nature for reinforced structures, except where alumina cement is used. Railway specifications in Germany[2] (AMB) rule out the use of water containing more than 3 percent sodium or magnesium chloride, so that the use of Baltic Sea water (total salt content 0.7 percent) and North Sea water (a total salt content of 3.3 percent, but sodium or magnesium chloride contents below 3 percent) is still permissible.

The ACI Building Code[3] makes no specific reference to inorganic salts, or to sea water, except for prescribing the use of water free of injurious amounts of deleterious materials. On the other hand, sea water is ruled out for prestressed concrete.[4] Gilkey[5] does not object to the use of sea water in making

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concrete for reinforced structures, provided a reduction in strength is allowed for. The Portland Cement Association[6] also permits its use for either plain or reinforced work. The British code[7] permits the use of sea water, except when efflorescence is undesirable. The Russian code[8] prohibits the use of sea water in reinforced maritime structures in hot, dry regions in view of the danger of corrosion and efflorescence; the implication is that its use is not objected to in regions other than coastal, hot or dry.

pH NECESSARY TO INHIBIT CORROSION OF REINFORCEMENT

It is well known that the protection against corrosion afforded the steel by the mortar (or concrete) covering it, is due to its alkalinity, and in the presence of salts the alkalinity required for inhibition of corrosion is greater than in their absence. Since no adequate data on the pH values required under various conditions could be found in the literature, a study of these problems was included in the investigation.

Method of test

Former tests indicated that reinforcing steel behaves in concrete, with regard to corrosion, similar to steel in alkaline solutions. Hence, it was assumed that a study of submerged corrosion of steel in alkaline solutions may contribute considerably to understanding the behavior of steel in mortar and concrete. Composition and alkalinity range of the several solutions tested are given in Table 1. The pH values of the various alkaline media, as well as pH values of cement pastes and mortars under various conditions, were measured, the first by a Pye pH-meter with a lithium-glass electrode, the latter by a Beckman pH-meter with a blue glass electrode.

Reinforcing bars were pickled, washed, dried, and weighed,[9] and then put into glass containers filled with the solutions. Some of the glass containers were hermetically closed, others closed with a screw lid so that some air penetrated (assumed to resemble conditions in dense concrete), while still others were given intensive aeration by air free of carbon dioxide. The rust

TABLE 1—CHARACTERISTICS OF SOLUTIONS USED IN SUBMERGED CORROSION TESTS

Composition	Range of pH values at 25 C
$Ca(OH)_2$	10–12.65
$Ca(OH)_2$ + NaCl	10–12.60
Extract of concrete made with sea water and extracted with sea water	10–12.50
Cement extract (with sea water)	12.80

TABLE 2—QUANTITY OF RUST OF STEEL IMMERSED IN $Ca(OH)_2$ SOLUTIONS OF VARYING pH

pH value of solution	Quantity of rust, percent of bar weight Age at test, 3 months	
	Limited access of air	Aerated
10.0	0.74	2.46
10.5	0.81	2.20
11.0	0.30	2.00
11.5	0.10	1.80
12.0	0.02	0.02
12.65	0.02	0.01

development, its distribution, and character were observed during a period of up to 3 months. The specimens were then taken out of the solutions, cleansed as above, and reweighed.

Corrosion in $Ca(OH)_2$ solutions

To ascertain the value of the pH necessary for the inhibition of corrosion of steel by hydroxyl ions deriving from $Ca(OH)_2$ (present, as well known, in mortar and concrete) steel bars were immersed in $Ca(OH)_2$ solutions of various concentrations. Half of the specimens were put into full containers closed with screw lids (limited access of air), the other half into partly filled containers, into which CO_2-free air was bubbled in daily (herein after marked "aerated"). The resulting corrosion is reported in Table 2.

The data show that practically complete inhibition against corrosion was provided by calcium hydroxide solutions of pH $\geqq$ 12.0. The small amount of rust found at pH = 12 (less than 0.02 percent) was almost invisible "general" corrosion. The influence of aeration was considerable up to the pH value of 12.0 and disappeared entirely at that value. For very limited aeration (assumed similar to that in very dense concrete) pH = 11.5 may be considered sufficient to inhibit corrosion of steel.

Effect of salts on saturated $Ca(OH)_2$ solutions

The effect of NaCl and of sea salts on the pH value of a saturated $Ca(OH)_2$ solution has been measured. Because solid $Ca(OH)_2$—in excess of that in solution—is present in mortars and concretes, a solution with an excess of solid $Ca(OH)_2$ was used in the tests. Precautions were taken to prevent carbonation. The results presented in Fig. 1 show that sodium chloride and particularly sea salts reduce the pH of $Ca(OH)_2$ solutions.

Corrosion in extract of concrete and $Ca(OH)_2$ plus salt solutions

In the presence of salts a higher pH value is required[10] for inhibition, but no sufficient *quantitative* data could be found. To determine the required pH, a series of experiments was carried out, the variables being the pH values of extracts of concretes made with sea water and extracted with sea water,

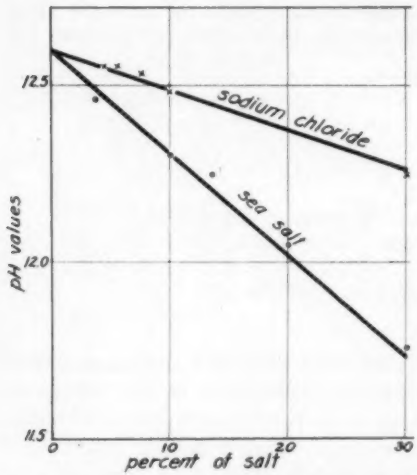

Fig. 1—Effect of salt concentration on pH values

and of calcium hydroxide solutions containing 3.6 percent sodium chloride. Results of these tests appear in Table 3.

On the basis of data in Tables 2 and 3, it is suggested that the pH values necessary for corrosion inhibition are as given in Table 4.

Character of the rust

As could be expected, the character of rust changed with the pH values and the composition of the solutions. In sea water extract of concrete, the rust changed with the pH value from large flakes covering whole bars (at pH = 10), through fine ones covering parts of bars, to localized rust at pH of about 12.0. Here the rust formed tubercles of a dark, almost black color, particularly at the ends of the bars. After the removal of the corrosion products deep pits became visible.

The localized corrosion reported here is consistent with the known action of anodic inhibitors, when present in an amount nearly sufficient to stop attack altogether.

At a still higher pH (12.75), which was obtained by extracting fresh cement with sea water, the bars remained practically free of corrosion, except for a thin greenish film. In calcium hydroxide solutions containing 3.6 percent sodium chloride the phenomena were similar, but the amount of rust larger, and the maximum pH value obtainable was still insufficient for complete inhibition under intensive aeration.

TABLE 3—RUSTING OF STEEL IN SOLUTIONS OF VARYING pH IN PRESENCE OF SALTS. TEST AGE, 3 MONTHS

pH value of solution	Rust, percent of bar weight, for exposure indicated			
	Extract of concrete*		$Ca(OH)_2$ + 3.6 percent NaCl	
	Limited air access	Aerated	Limited air access	Aerated
10.0	0.63	0.95	0.63	1.58
10.5	—	1.20	0.78	1.50
11.0	0.50	1.00	0.70	1.46
11.5	0.56	0.70	0.86	1.52
12.0	0.65	0.85	0.82	1.41
12.45	0.40	0.50	—	—
12.60	0.04	0.30	0.07	1.08
12.75†	0.04	0.02	—	—

*Made with sea water and extracted with sea water.
†Extract of portland cement in sea water.

TABLE 4—pH VALUES FOR INHIBITION OF CORROSION

Solution	$Ca(OH)_2$		Concrete extract in sea water		$Ca(OH)_2$ + 3.6 percent NaCl	
Aeration	Limited	Intensive	Limited	Intensive	Limited	Intensive
pH inhibition level	≧ 11.5	12.0	12.6	12.75	12.6	More than obtainable

On the other hand, in salt-free solutions, other conditions being equal, while the character of rust was not unsimilar, the pH values corresponding to the beginning of localization and to inhibition level were lower.

These values, both in salt-free and in salt-containing solutions, depend on aeration; the stronger the aeration, the higher the respective pH values, until a certain level is reached at which aeration has practically no effect. It is suggested that this level be called "complete inhibition level."

Influence of salt concentration on corrosion

One series of tests was devoted to the effect of salt concentration on corrosion. Sodium chloride in various amounts was added to $Ca(OH)_2$ solutions, in which steel bars were then submerged for 1 to 3 months and the quantity of rust determined. The results are given in Table 5.

The data in Table 5 show that with limited air penetration the increase of concentration of NaCl in calcium hydroxide solutions has a limited effect up to a certain maximum, above which a higher concentration restrains corrosion (for schematic presentation see Fig. 2). Akimov[12] mentions a similar trend of salt effect in neutral solutions. The explanation is that as the salt amount in the solution becomes greater the increase of the conductivity of the solution ceases, while the solubility of the oxygen keeps diminishing until at a certain concentration the shortage of oxygen has a predominant effect.

The influence of concentrations of 0.1 to 5 percent of NaCl in $Ca(OH)_2$ solutions of an initial pH of 12.6 was tested also in hermetically closed containers. In this interval the effect increases steadily.

TABLE 5—SODIUM CHLORIDE EFFECT ON RUST DEVELOPMENT IN $Ca(OH)_2$ SOLUTIONS OF VARYING pH

NaCl, percent	0	0.1	1.0	2.0	3.6	4.5	5.0	5.5	7.5	10.0	30.0
Limited air penetration											
Rust at 1 month, percent of bar (initial pH = 11.3)	0.31	—	0.56	0.54	0.58	—	—	—	—	0.12	0.09
Rust at 3 months, percent of bar (initial pH = 12.6)	0	—	—	—	0.04	0.18	—	0.07	0.05	—	—
Limited penetration of air free of CO_2											
Rust at 1 month, percent of bar (initial pH = 12.5)	0	0.01	—	0.04	—	—	0.03	—	—	0.03	0.01

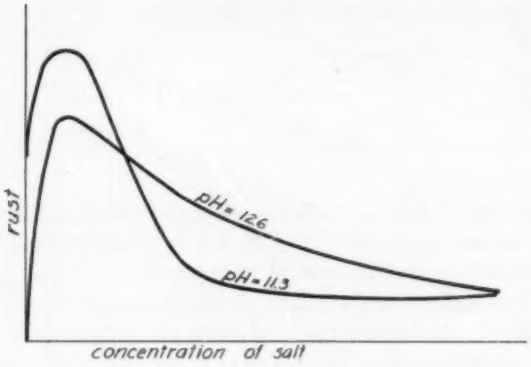

Fig. 2—Schematic presentation of the effect of salt concentration and pH in $Ca(OH)_2$ solution on rust development

pH VALUES OF PASTES AND MORTARS

The pH values of fresh cement pastes, as well as of fresh and hardened mortars of various consistencies and cement contents were measured. The results are given in Table 6. Fresh cement pastes mixed with tap water showed an average pH of 13.2, while that mixed with sea water showed pH = 13.0.

The pH values of fresh cement mortars were affected by the quality of the mixing water and by consistency. No clear-cut effect of cement content was found, but it could be established that the effect is very small, if any.

The pH values of mortars stored in closed vessels for 28 days, ground, and measured at that age were also affected by the quality of mixing water. The effect of the initial consistency disappeared. Practically no effect of cement content for 1:10 to 1:1 mixes was found. Mortars made with tap water had an average pH of 12.8, and for those made with sea water pH $\leqq$ 11.8.

Mortar specimens made with sea water and stored 28 days in sea water showed a pH value of 12.95.

TABLE 6—pH OF PASTES AND MORTARS

Cement-aggregate ratio	Mixing water	Consistency	Age at test	Average pH
1:0 (paste)	Tap	Semidry to very wet*	½ hr	13.2
	Sea	Semidry to very wet*	½ hr	13.0
Ranging from 1:10 to 1:1†	Tap	Semidry	½ hr	12.5
		Plastic	½ hr	12.8
		Very wet	½ hr	12.9
	Sea	Semidry	½ hr	12.0
		Plastic	½ hr	12.1
		Very wet	½ hr	12.2
Ranging from 1:10 to 1:1†	Tap	Semidry to very wet‡	Closed storage, 28 days	12.8
	Sea	Semidry to very wet‡	Closed storage, 28 days	≦ 11.8
			28 days (stored in sea water)	12.95

*No special effect of consistency reported.
†Little or no effect of cement content was found.
‡Effect of initial consistency disappeared.

TABLE 7—PROPERTIES OF REINFORCEMENT

Chemical components	Percent	Mechanical properties
C	0.10	Yield point—44,600 psi
P	0.05	Tensile strength—58,200 psi
S	0.07	Percent elongation—20
Mn	0.40	Percent reduction in area—67
Cu	0.04	

TABLE 8—COMPOSITION OF THE MIXING WATER

Compound	Grams per liter
KCl	0.863
K_2SO_4	0.863
$MgBr_2$	0.076
$MgCl_2$	3.810
$MgSO_4$	1.660
NaCl	27.200
$CaSO_4$	1.260
Total	35.732

STUDY OF REINFORCED MORTAR PRISMS

The present study of the formation and development of rust in mortars containing salts is the sequel to an investigation by one of the authors of reinforcement corrosion in salt-free mortar and concrete, published in an earlier JOURNAL.[9] Variables studied were cement content, water-cement ratio, and consistency. It early became evident that interpretation of the results would be most difficult unless more fundamental data on inhibition of corrosion by mortar and concrete were available. Hence the foregoing tests of corrosion inhibition level and pH of paste and mortar were undertaken.

Specimens and test procedure

The more than 400 mortar specimens tested from 1953 to 1957 were prisms 4 x 4 x 16 cm, reinforced with 6 mm mild steel bars whose properties are given in Table 7. Depth of cover was about 1.7 cm. All specimens were mixed with Mediterranean Sea water (properties in Table 8). Chemical and mechanical properties of the cement used are listed in Table 9. Aggregate for the mortar (Table 10) was composed of 65 percent (by weight) coarse

TABLE 9—PROPERTIES OF CEMENT

Chemical components	Percent	Mechanical properties (ASTM C 150-53)	
SiO_2	20.0		
Al_2O_3	6.5	Specific area (Blaine)	3450 sq cm per g
Fe_2O_3	3.2	Tensile strength	3 days 375 psi
CaO	63.3		7 days 425 psi
MgO	1.7	Compressive strength	3 days 2600 psi
SO_3	2.0		7 days 3500 psi
Loss on ignition	2.2		
Nonsoluble	0.7	Initial setting time	130 min
Alkalies	0.4	Soundness test	0.34 percent
	100.0		

TABLE 10—PROPERTIES OF AGGREGATES

Compound	Chemical composition, percent		Mechanical analysis		
	Coarse sand	Fine sand	Sieve size (British)	Percent retained	
				Coarse sand	Fine sand
$CaCO_3$	81.0	14.0	No. 7	0	
$MgCO_3$	8.0	Traces	No. 14	5.5	
SiO_2	3.5	84.0	No. 25	65.4	0
$Al_2O_3 + Fe_2O_3$	7.5	1.0	No. 52	99.4	2.7
NaCl	Traces	Traces	No. 100	100.0	83.2
			Fineness modulus	2.70	0.86
			Unit weight, kg per cu m	1500	1400

TABLE 11—CHARACTERISTICS AND STORAGE CONDITIONS FOR MORTAR PRISMS

Cement-aggregate ratio, by weight	Water-cement ratio, by weight	Consistency	Reinforcement	Storing	Age at test, months
1:2	0.5, 0.65, 0.8	Semiplastic to sloppy	Pickled*	Moisture room	3, 6, 12 24, 48
1:4	0.5, 0.65, 0.8 0.95, 1.1	Semidry to sloppy	Pickled Unpickled rusty	Moisture room	3, 6, 12 24, 48
1:6	0.5, 0.65, 0.8 0.95, 1.1	Semidry to plastic	Pickled	Moisture room Sea water Tap water	3, 6, 12 24, 48 6 6
1:8	0.65, 0.8 0.95, 1.1	Semidry to semiplastic	Pickled	Moisture room Sea water Tap water	3, 6, 12 24, 48 6 6
1:10	0.65, 0.8 0.95, 1.1	Very dry to semiplastic	Pickled	Moisture room	3, 6, 12 24, 48

*Pickling process as described in Reference 9.

sand and 35 percent fine sand, with an over-all fineness modulus of 2.06. Characteristics of the test specimens and conditions of storage are given in Table 11.

The steel bars were pickled, washed, dried, and weighed (as described in detail in Reference 9) before being inserted in the mortar specimens. Control specimens with unpickled bars, some of them particularly rusty, were also tested. The mortar specimens were broken at ages from 3 to 48 months. The bars were observed, cleansed as above, and reweighed—weight loss indicating the amount of rust. Depth of pits and area of pitting were also measured.

General observation of specimens after storage in moist room

The specimens were damp to the touch compared with similar specimens made with tap water (due to the presence of the hygroscopic salts).

The reinforcement was corroded in all specimens stored in the moist room, much more so than in salt-free mortar under similar conditions.[9] The difference was particularly pronounced in rich mortars. In Fig. 3 some samples of the corroded bars are shown.

Most of the rust was dark, almost black (magnetite?), damp and semisolid, in contrast with the reddish brown color (ferric oxide) characteristic of reinforcement embedded in mortar free from salts.

To clearly demonstrate the effect of sea water versus tap water, turned steel bars were coated with 1 mm thick cement pastes made with sea and tap water respectively, both with the same W/C ratio (0.63). The rust products, which penetrated through the coat, are shown in Fig. 4.

After removal of the bars from the mortar specimens, the rust remaining in the grooves was found to be mainly brown and partly black, and it may be assumed that the brown rust was formed from the black one in a secondary

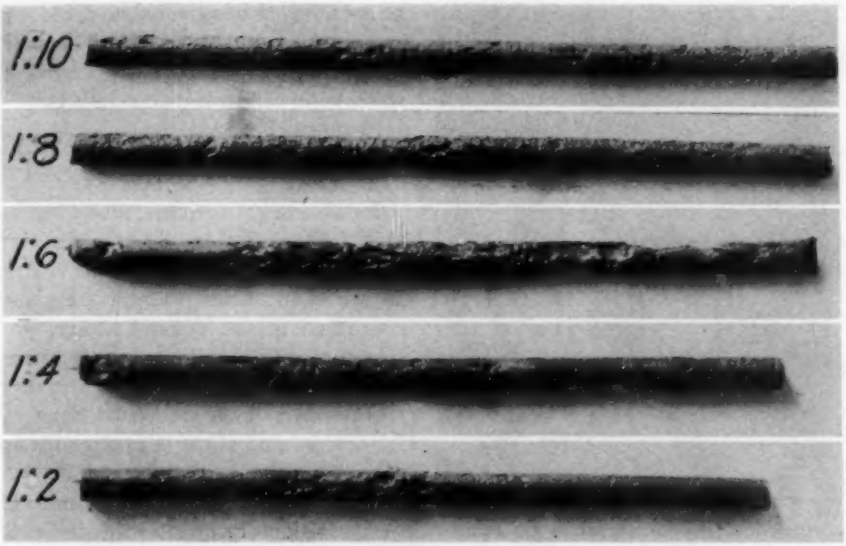

Fig. 3—Samples of corroded bars after removal from mortar test specimens. Numbers indicate cement aggregate ratio of mortar

reaction. Migration of the rust was observed within the prism, sometimes reaching the surface. After removal of the rust from the bars, extensive pitting was found, particularly so under the black rust.

The distribution of rust on the bars was uneven, especially in rich mixes. Accumulation of corrosion products was observed at the edges of the bars. Parallel tests with unpickled bars showed that the character, the uneven distribution, and the color of the rust, as well as the degree of pitting, were similar. The same was observed in another case of unpickled steel, already considerably corroded before use.

The average amount of rust in 1:2 mortar was small. Few, but deep, pits were formed and the rust, concentrated at the edges, led to cracking and spalling of the prisms (Fig. 5). Similar cracks appeared also in 1:4 mortar,

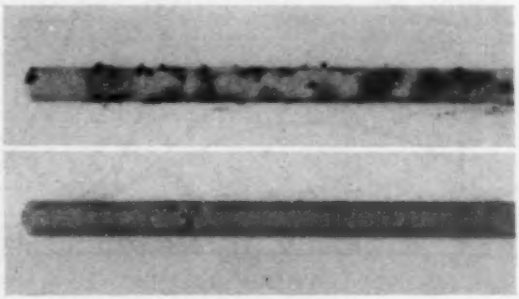

Fig. 4—Rust on turned bars coated with cement paste (W/C = 0.63) made with sea water (above) and tap water (below) after 1 month in moisture room. Dark spots show rust products

Fig. 5—Flaking off and cracks caused by corrosion of reinforcement in 1:2 mortar

to a smaller degree in 1:6, and did not appear at all in 1:8 and 1:10 mixes. Pitting was deep in 1:4 and 1:6, shallow in 1:8, and especially so in 1:10 mortar.

Evaluating the degree of corrosion

Establishing a quantitative criterion for rusting in the presence of salts is a complicated problem. First, as stated by Evans[10] and others, and confirmed in the present study, pitting varies in depth, area, etc. from specimen to specimen, even under theoretically identical conditions of formation, and there is practically no reproducibility of results to be expected. Secondly, the amount of rust, satisfactory criterion in the case of general corrosion, is not adequate in the case of pitting. Deep pits often appear in largely rust-free bars. Thus in spite of the low average rust content, the local reduction in area may be high.

Despite these limitations of the weight-loss method in the case of pitting, it was considered worthwhile to include the data on weight loss, as indicative of certain typical phenomena (see Table 12).

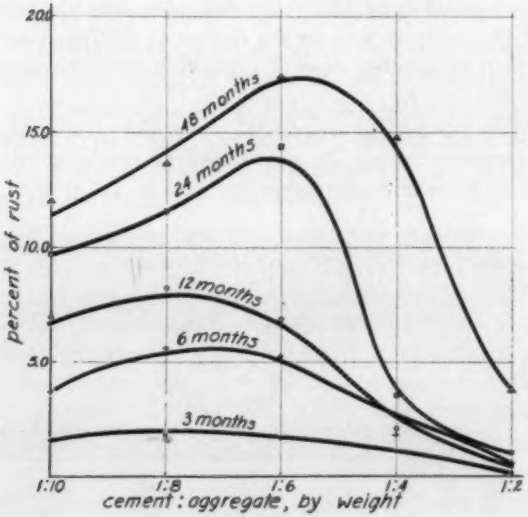

Fig. 6—Relation between cement content of salt-containing mortar and rate of rusting of reinforcement, moisture-room storage

To obtain fuller data, depth of pits and area of pitting* were also measured. Ratings from 0 to 4 were assigned to the corroded bars: 0 for a bar free of pitting, and 4 for a bar pitted over its entire surface. The results at 48 months are given in Table 13.

Test results

The following phenomena regarding specimens stored permanently in the moisture room were particularly conspicuous:

1. Corrosion in salt-containing mortars intensifies with time both in area and in depth, irrespective of cement contents, water-cement ratios, and consistencies tested, except for the 1:2 mix with an almost sloppy consistency ($W/C = 0.80$) where it seems to come to a standstill after 2 years (Table 12).

TABLE 12—AMOUNT OF RUST, PERCENT OF BAR WEIGHT, OF BARS EMBEDDED IN MORTAR MIXED WITH SEA WATER, STORED IN MOIST ROOM

W/C of mortar	Age, months	Amount of rust, percent of bar weight, for given cement-aggregate ratio of mortar				
		1:10	1:8	1:6	1:4	1:2
1.1	3	1.6	1.0	0.9	0.3	—
	6	3.7	2.3	2.6	1.5	—
	12	7.0	9.0	3.3	2.4	—
	24	9.7	6.7	13.0*	3.1	—
	48	12.4	16.2	19.1	14.0	—
0.95	3	1.1	1.9	1.7	0.9	—
	6	2.6	5.6	5.2	1.6	—
	12	3.7	8.2	8.1	3.9	—
	24	7.3	11.6	14.4	5.3†	—
	48	9.4	13.7	17.4	5.9‡	—
0.80	3	2.5	1.5	1.8	1.1	0.6
	6	5.0	1.9	4.2	1.8	0.9
	12	6.3	2.9	10.4	2.0	0.6
	24	8.9	7.2	12.3	3.6	1.4†
	48	10.9	13.0	15.9	14.9	1.2†
0.65	3	0.9	2.1	2.5	4.5	0.1
	6	2.2	5.3	3.2	4.6	0.6
	12	2.7	5.8	5.5	6.9	0.6
	24	7.9	10.0	9.5	11.7	1.0
	48	10.2	13.0	17.7	14.5	3.9*
0.50	3	—	—	3.7	3.9	2.2
	6	—	—	4.2	4.9	1.8
	12	—	—	5.9	6.7	4.3
	24	—	—	9.3	9.7§	5.7
	48	—	—	12.3	9.6§	7.8

*One prism cracked by rust.
†Two prisms cracked by rust.
‡All prisms cracked by rust.
§Average of scattered values of 11.6, 11.1, and 6.06 percent.

TABLE 13—PITTING OF BARS EMBEDDED IN MORTAR MADE WITH SEA WATER, STORED IN MOIST ROOM FOR 48 MONTHS

W/C of mortar	Pitting rating for bars in mortar of given cement-aggregate ratio				
	1:2	1:4	1:6	1:8	1:10
0.50	1.2	2.8	2.3	—	—
0.65	0.3	2.4	2.6	2.7	2.0
0.80	0.4	2.6	1.9	2.5	2.4
0.95	—	0.9	2.3	2.3	1.6
1.10	—	1.9	1.9	1.8	2.2

*Spongy corrosion is not included in this term.

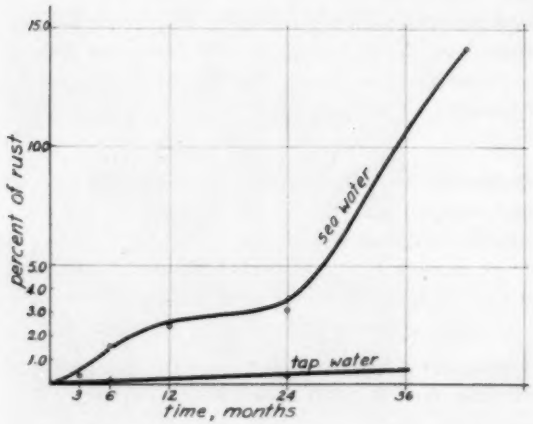

Fig. 7—Time-rust relation in mortar 1:4 mixed with tap and sea water, respectively

Reinforcement in salt-free similar mortars behaves in a markedly different way (Fig. 7). A few comparative data are given in Table 14.

In a parallel test made with unprotected exposed steel, stored in the same moisture room, the rate of corrosion tended to decrease asymptotically with time and the total amount of rust was small (Fig. 8).

2. The amount of rust and the area and depth of pitting were not governed by the water-cement ratio, as can be seen from Tables 12 and 13. This is also shown in Fig. 9 where the bar coated with 1 mm thick cement paste mixed with sea water is almost wholly covered with rust when $W/C = 0.27$, while with $W/C = 0.63$ there are only rust stains to be seen. Parallel specimens in salt-free cement paste are shown in Fig. 10. Here the effect of the cement coat was similar in the sense that the wetter paste afforded better protection, but both bars corroded, of course, much less.

3. No regular relationship was observed between consistency and amount of rust, in contrast to salt-free mortar.[9] Fluid mortars, however, showed advantage over drier mixes, other conditions being equal.

4. Statistical analysis* of data in Table 13 shows that the pitted area

*The pooled variance within samples was $s^2 = 0.098$ with 34 degrees of freedom. The lowest significant difference between any two sample means was therefore 0.433, for $P = 0.95$ level of significance. Sample means were separated into homogeneous groups by the method of D. B. Duncan (1955).

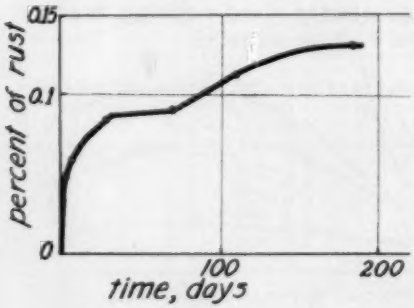

Fig. 8—Time-rust relation of unprotected steel stored in moisture room

TABLE 14—RUST IN SALT-CONTAINING AND SALT-FREE MORTARS, STORED IN MOIST ROOM

Cement-aggregate ratio	Consistency	Age at test, months	Rust, percent of bar weight, in mortar made with	
			Tap water	Sea water
1:4	Plastic	3	0.04	0.70
1:4	Plastic	6	0.37	1.36
1:4	Sloppy	24	0.27	3.10
1:4	Sloppy	48	0.63	14.00

depended on consistency (as measured by increasing W/C ratio) for a mix of given cement-aggregate ratio. In all mixes reinforcement in mortars of a dry consistency showed a higher degree of pitting. The increase in water content considerably reduced the pitted area in 1:2 mortars, less so in 1:4 and 1:8. No monotonic reduction was found in 1:6 and 1:10 mortars, though here too the same tendency seems to exist.

5. No regular relationship between the cement content and amount of rust has been found, though the 1:2 mortar was the best in all cases. Yet only at sufficiently fluid consistencies (W/C = 0.65 and 0.8) was the amount of rust low; at a semiplastic one (W/C = 0.50) it was considerable.

The 1:4 mortar provided adequate protection to the reinforcement for a limited period only (in the present tests for 2 years), after which extensive corrosion set in. The 1:6 mortar, at all water-cement ratios, showed no advantage over 1:8 and 1:10 mixes and was actually inferior in some cases, as is shown by the rust quantities after 2 and 4 years. At nearly all W/C ratios, 1:10 mortar was superior to the 1:8 mix.

6. In Fig. 11 weight of rust is plotted against degree of pitted area. It can be seen that in the 1:2 mortar the increase in weight goes together with increase of area. In other mixes the scatter of the data indicates that the increase in weight from 10 to 20 percent is caused by an increase in the depth of pits, rather than in area.

7. Maximum pitting depth in all mortars, with the exception of the 1:10 mix, was from 1.7 to 2.1 mm. In many bars the pits formed opposite one another, reducing the bar diameter from 6 to about 3.5 mm. In the 1:10 mix, pits were few and rather shallow.

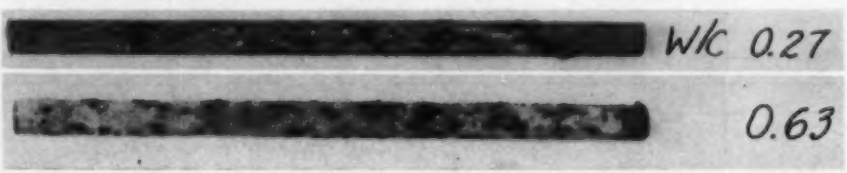

Fig. 9—Rust on bars coated with cement paste of water-cement ratio of 0.27 and 0.63 respectively, made with sea water, after 1 month in moisture room. Dark spots show rust products, light areas the cement paste

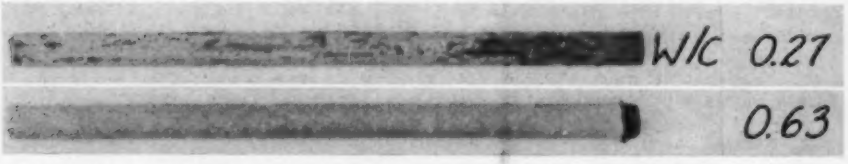

Fig. 10—Rust on bars coated with cement paste of water-cement ratio of 0.27 and 0.63, respectively, made with tap water, after 1 month in moisture room. Dark spots show rust products, light areas the cement paste

Comparison with other storage conditions

All of the foregoing data refer to specimens stored in a moisture room. For comparison, parallel specimens from a number of mixes were stored in tap and sea water. Results are given in Table 15. The data show clearly that in the case of completely immersed specimens, the salts present either in the mixing or in the storing water do not affect the reinforcement, and that the steel behaved just as in salt-free mortars.

DISCUSSION OF RESULTS

The data obtained by submerged corrosion tests in calcium hydroxide solutions and concrete extracts are considered an appropriate background, against which the results of the study of behavior of reinforcement embedded in various mortars should be discussed.

First, it has been found that in the presence of salts none of the mortar mixes tested afforded adequate protection to the reinforcement unless permanently immersed in water. This is understandable in the light of the pH values required for inhibition of corrosion as presented in Table 4, on the one hand, and of the pH values of the salt-containing mortars, as measured, on the other. The pH values of the air-stored mortars were below the level of complete inhibition and consequently corrosion set in.

Amount and character of corrosion

In most specimens the pH was close to the inhibition limit, a fact which explains why the corrosion took the form of pitting. In the very porous mortar of the 1:10 mix, where the pH was considerably reduced by carbona-

TABLE 15—INFLUENCE OF STORAGE CONDITIONS ON CORROSION AT TEST AGE OF 6 MONTHS

Cement-aggregate ratio	1:8			1:6									
W/C	1.1			1.1			1.1	1.3		0.8		0.6	
Consistency	Plastic			Very plastic			Very plastic	Very sloppy		Stiff-plastic		Semidry	
Mixing water		SW			SW		TW	TW		TW		TW	
Storing	MR	SW	TW	MR	SW	TW	MR	SW	TW	SW	TW	SW	TW
Rust, percent of bar weight	2.2	0.07	0.09	1.4	0.08	0.04	0.22	0.01	0.01	0.02	0.01	0.02	0.01

MR = moisture room; SW = sea water; TW = tap water.

tion, reaching presumably a much lower level than in the richer mixes, the corrosion was of the "general" type.

In addition to the alkalinity of the environment, as measured by the pH values, the amount and character of corrosion of steel in mortar or concrete are affected by such factors as differential variation of the salt concentration, the moisture, the aeration (CO_2 and oxygen), etc., which induce the formation of microcells. The combined effects brought about a variety of phenomena, as observed in the tests reported here.

Influence of cover

The present investigation confirmed the conclusion drawn in the former study[9] that evenness of cover over the reinforcement plays a pronounced part in protection against corrosion. This evenness of the covering proved more decisive than its density, as shown by the fact that wet mortars provided better protection. This may partly be due to the higher initial pH values of wet mortars.

Effect of storage conditions

Specimens stored permanently in water had the double advantage of a pH value above the inhibition level (12.95) and a uniform salt concentra-

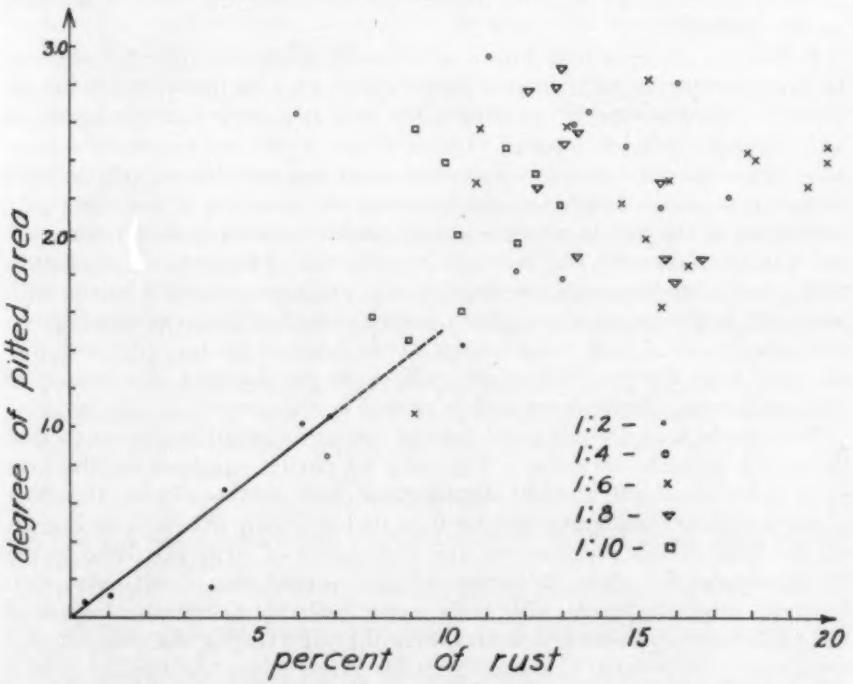

Fig. 11—Relation between percentage of rust and degree of pitted area in different mortar mixes

tion.* This explains the fact that in mortar made with sea water and immersed in tap or sea water, practically no corrosion of reinforcement took place (there is no fear of an increase in the concentration of the soluble $CaCl_2$ formed by the action of $MgCl_2$, present in the sea water, on the $Ca(OH)_2$ of the mortar; an increased $CaCl_2$ concentration would have lowered the pH considerably).

Although the tests on which this last conclusion is based were conducted over a period of 6 months only, the results can be accepted as permanently valid—in view of the prevalence of the basic conditions (uniform salt concentration and sufficient alkalinity) as long as the dense concrete, in a state of complete and uninterrupted submersion, does not deteriorate. In the case of air-exposed concrete, however, the data indicate that only long-term tests can provide conclusive results (see Table 12).

Concrete behavior inferred from mortar tests

The authors suggest that although the test data are confined to mortars, inference may also be drawn as to the behavior of reinforcement in concrete. This suggestion is based, among others, on the former study[9] on corrosion of reinforcement in salt-free mortar and concrete, which furnished ample evidence, that the behavior of steel embedded in mortar is basically the same as when embedded in concrete, though quantitatively the effect is not necessarily identical.

It may be assumed that whenever moisture penetrates into dry salt-containing concrete the salts dissolve immediately, whereas the sparingly soluble $Ca(OH)_2$ dissolves slowly, yielding a low pH at a high salt concentration and thus facilitating corrosion. This, of course, would not happen in a specially dense concrete (the kind of concrete which can be achieved only by close attention to every detail in connection with the selection of materials, proportioning of the mix to produce a truly plastic concrete having a relatively low water-cement ratio, and thorough consolidation of the concrete as placed). Still, even when moisture penetration into reinforced concrete mixed with sea water is prevented, the question remains whether the hygroscopic moisture (composed of both some mixing water retained by the salts and that absorbed from the penetrating air) suffices for development of corrosion of any significance. Further research is needed in this respect.

Tests made with $Ca(OH)_2$ solutions of low salt concentrations prove that they, too, promote corrosion. This may be partly explained by the lowering of the initial pH, affected also by small concentration of salts, the effect of sea salt being considerably greater than that of sodium chloride (see Fig. 1). On the basis of this phenomenon and observation of structures close to the Mediterranean Sea shore, it is deduced that penetration of salt spray into concrete (originally made with fresh water) is likely to cause corrosion of the reinforcement, because it both lowers the pH value of the concrete and introduces a differential salt concentration. Here again the remedy is in a very dense concrete or a suitable waterproof coating.

*In specimens stored in the moisture room differential salt concentration was measured.

Atmospheric carbonation effect

One more point warrants explanation. In the laboratory tests CO_2-free air was used for aeration of the $Ca(OH)_2$ solutions, to avoid reduction of their alkalinity by carbonation. Now, what about the effect of atmospheric carbonation on the pH of mortar and concrete?

There is enough evidence to show that in a concrete of ordinary quality atmospheric carbonation is not likely to proceed beyond a surface layer up to a few millimeters thick;[13,14] thus the pH of more than half of the usual depth of cover and particularly of the immediate environment of the reinforcement would remain unaffected, similarly to the pH of $Ca(OH)_2$ solutions into which CO_2-free air was introduced. The few tests made by the authors on freshly cut mortar and concrete surfaces by treating with titan yellow and tropeolin 0 proved this presumption to be correct.

CONCLUSIONS

On the basis of the above tests and considerations the following conclusions are suggested:

1. The alkalinity of air-exposed mortars and concretes made with Mediterranean Sea water is insufficient for complete inhibition of corrosion of reinforcement. This is also the case when sea salts penetrate into concrete made with tap water.
2. Sea water used for mixing mortar or concrete for air-exposed reinforced structures tends to make the reinforcement highly vulnerable to corrosion.
3. Maritime reinforced structures should be protected against any penetration of sea water or salt spray by using specially dense* concrete and adequate depth of cover, or by means of special surface rendering.
4. It is believed that the laboratory test data corroborated by the behavior of sea structures in practice, constitute sufficient ground for the supposition that there is no danger of corrosion of reinforcement in concrete made with tap or sea water in the case of permanent and total immersion in sea water, provided the usual requirements for concrete made to withstand salt water, waves, etc., are observed.

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Title No. 55-77

Waterstops for Joints in Concrete

B. KELLAM and M. T. LOUGHBOROUGH

Waterstops are grouped into six categories, depending on shape and material. Studies have indicated that a properly compounded polyvinyl chloride is likely to outlast many of the materials used in the past.

An investigation of the watertightness of various shapes and sizes of waterstops is described, from which it is concluded that "flat corrugated" and "flexible metal" types are superior to "dumbbell" and "metal plate" waterstops.

Physical properties of waterstops are discussed with reference to their ability to withstand rough treatment during installation and their ability to accommodate joint movements. Waterstops of various shapes and materials are compared with regard to ease of installation. Methods developed for cutting, splicing, and installation of pvc waterstops are outlined.

■ JOINTS IN CONCRETE STRUCTURES, one side of which are subjected to hydrostatic load, must contain special provisions to insure watertightness. Such joints are generally provided with waterstops consisting of a diaphragm of metal, rubber, or plastic, bridging the joint and embedded in the concrete on either side. In a contraction joint where expansion and contraction of the concrete cause the joint to close and open, the waterstop must be designed to accommodate itself to movement. A construction joint in which no movement is expected to occur, does not impose this requirement on the waterstop.

Prior to World War II most waterstops were made of metal and consisted of either a rigid metal plate or some form of flexible metal diaphragm. In 1935, the U. S. Bureau of Reclamation instituted the use of rubber as a material for waterstops. These waterstops consisted of dumbbell shapes and were used as a substitute for metal in various applications. Rubber waterstops of the same shape are still being used and, in general, appear to be giving satisfactory service.* With the rapid progress in the development of plastics during and after World War II, manufacturers have recently marketed waterstops of various shapes made of polyvinyl chloride (pvc).

*Allen, E. A., and Higginson, E. C., "Waterstops in Articulated Concrete Construction," ACI JOURNAL, V. 27, No. 1, Sept. 1955 (*Proceedings* V. 52), pp. 93-102.

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Up to 1954, The Hydro-Electric Power Commission of Ontario used only metal waterstops. Although these have performed well in most instances, some cases of leakage have occurred and it was felt that an investigation into the merits of the flexible waterstops of rubber and pvc was warranted.

The investigation was designed to cover as many aspects of the problem as possible. All known types of waterstops were considered and the most commonly used were subjected to leakage tests at various water pressures and joint openings. Other service requirements such as longevity, strength, and elongation were investigated. Tests were run on the handling properties such as cold weather flexibility, the resistance of waterstops to rough handling on the job, and ease of installation. On completion of the investigation the decision was made to adopt plastic waterstops and a further study was made of installation techniques, including methods of cutting and splicing. Finally, a specification covering materials and shapes was developed for use in the purchase of waterstops for Ontario Hydro use.

TYPES OF WATERSTOPS

Types of waterstops investigated logically fall into six groups designated here as labyrinth,* flat corrugated,* dumbbell 2-bulb, dumbbell 3-bulb, flexible metal, and steel plate. Fig. 1 shows a typical example of each type. The first four are manufactured from rubber or polyvinyl chloride. The labyrinth type was designed to obviate the need for splitting the forms to install the water-stop and is limited in its use to construction joints where little or no movement is expected. The flat corrugated type is characterized by numerous ribs or corrugations running the length of the waterstop. The dumbbell types rely on the bulbs at the ends of the section to provide anchorage in the concrete, and the hollow center bulb in the 3-bulb type makes provision for shear movement in the joint. The flexible metal type may take many forms. The typical one shown is a strip of thin gage metal, often copper or stainless steel, with a deep crimped V-groove in the center to accommodate joint movements. The steel plate type refers to a heavy

*Patented.

gage flat metal plate, one-half of which is embedded firmly in the concrete on one side of the joint, the other half being coated with asphalt to allow it to slip in the concrete when movement occurs.

DURABILITY OF WATERSTOP MATERIALS

Since in many concrete structures the replacement or repair of faulty waterstops is a prohibitive undertaking, it is of paramount importance that a waterstop should continue to perform well for the life of the structure. Massive structures such as dams may conceivably be required to last for several centuries. How will the various materials of which waterstops are made, stand up to such long periods of service?

Metal

Corrosion is likely to be the predominant factor with wrought iron or carbon steel. On the upstream face in the vicinity of the water line, and on the downstream face, where the air in contact with the waterstop is likely to be moist, the prevailing conditions are likely to result in a high rate of corrosion. To estimate life expectancy would be hazardous, since so much depends on conditions and on the effectiveness of any protective coating applied to the water-stop. So far as corrosion is concerned, copper or stainless steel might be expected to last indefinitely. With copper waterstops mechanical fatigue caused by repeated opening and closing of the joint may be the cause of failure rather than corrosion.

Rubber

Rubber is longest lived completely out of contact with oxygen, and when protected from sunlight. Waterstops are generally not subjected to sunlight but do contact oxygen in air on the downstream side and in water on the upstream face. The degradation of rubber exposed to oxygen causes hardening and eventual cracking. If rubber is completely relaxed the rate of degradation is much slower than when under stress, particularly tensile stress.

The oxygen-bomb test is a standard test for determining the resistance of rubber to aging. Unfortunately, there is no known correlation between the life of rubber for any application and the results of the oxygen-bomb

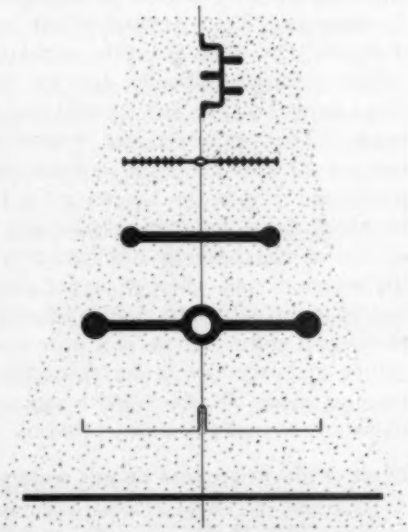

Fig. 1—Examples of six types of waterstop. From top to bottom: labyrinth, flat-corrugated, dumbbell 2-bulb, dumbbell 3-bulb, flexible metal, and steel plate

test. This is due to the large number of variables in each application that can and do affect service life. The test, therefore, is useful only for comparing the resistance of one rubber with that of another.

From the viewpoint of resistance to alkaline environment, rubber has a high order of durability. It will be of the same order of durability as pvc. It is not subject to extraction of plasticizer like pvc; the rubbers tested to date have all shown a gain in weight on immersion in laitance water.

The longest record known to the authors of the use of rubber waterstops on this continent is that of the U. S. Bureau of Reclamation, mentioned earlier, dating back to 1935. Field observations of rubber used in other applications where air is a contact medium, suggest that 35 years is the longest period for which satisfactory performance can be anticipated.

Polyvinyl chloride

Polyvinyl chloride, when pure and uncompounded, forms articles that are hard and rigid. When compounded with certain plasticizers, commonly liquids, pvc becomes flexible, tough, and leather-like. Ontario Hydro has had experience with pvc for conductor insulation and sheathing dating back to 1940. The pvc has been exposed under a variety of conditions, much of it underground, and without a single failure due to deterioration.

Extensive evaluation of plasticized pvc used for insulation has shown that it is virtually unaffected chemically by alkaline environment much stronger than that associated with freshly-placed concrete. This means that there is no chemical deterioration of the pvc as a result of embedment in concrete. Tests performed on waterstop vinyls, the first of which was done early in 1952, show the same chemical inertness.

The difference between pvc for electrical insulation and that for waterstops is in the type and quantity of plasticizers used. The base resin is the same. That approved for waterstops is especially compounded to allow use over a wide range of temperatures. A common weakness of ordinarily plasticized pvc is the tendency for the plasticizer to migrate to any liquid in which the compound is submerged. When the liquid is water the rate of loss of plasticizer is influenced by the water temperature, the nature of the solution, and whether or not the water is periodically renewed. If this loss is appreciable the pvc gradually reverts to its original stiff, hard, unplasticized state. There is a wide variety of plasticizers of varying plasticizability and behavior under extraction conditions. In recent years, pvc has become available specially compounded with polymeric plasticizers, and these are virtually unextractable by water.

Effect of plasticizer loss on pvc waterstops

To assess the effect of plasticizer loss on the ability of the pvc to function as a waterstop, a test was developed that was later incorporated in Ontario Hydro's specification for pvc waterstops. ASTM dumbbell-shaped tensile-test specimens were cut from $\frac{1}{8}$ in. thick sheets and weighed. These were immersed in a solution of 0.5 percent sodium and 0.5 percent potassium

hydroxides stirred by bubbling air. The content and concentration of the extracting solution was based on the analysis of the strongest concrete laitance water encountered in two decades of study, and it approached the limit of the pH scale on the alkaline side. This concentration is believed to be the strongest to which a waterstop will be subjected in service. The solution was kept at 140-150 F and changed each day. The test was continued until daily loss in weight of the specimens reached a minimum. With materials tested to date the time required has varied from 17 to 82 days and the loss from 0.14 to 7.39 percent; maximum gain was 0.47 percent.

After this treatment the specimens were tested in tension and elongation, the results being compared with the "as-received" values. Reduction in tensile strength and elongation ran as high as 16 percent and 30 percent, respectively, but the residual values were considered adequate. In this way the performance of a waterstop material was determined under the condition where all extractable plasticizer was lost—a condition that would take many years to reach in service.

Another consequence of the loss of plasticizer was the possibility that such organic matter might be deleterious to freshly placed concrete, in line with the generally accepted belief that organic matter in concrete is undesirable. For this reason the acceptance tests originally included one in which specimens cut from the waterstop, or from molded sheets, were immersed in the same alkaline solution as above for 30 days at room temperature. Changes in weight, durometer hardness, and dimensions were noted at 7 and 30 days. An arbitrary maximum weight gain permitted at 7 and 30 days was 0.25 and 0.40 percent, respectively. No loss was permitted. The maximum permissable change in durometer hardness was ± 5 points.

Additional tests, run since Ontario Hydro's specification was first drawn up, involved exposure of freshly hardened mortar trays to several of the more common plasticizers in the pure state. After 1 month, examination of exposed surfaces showed no discernible change when compared with the control specimens. Accordingly, the specification has been amended to permit a decrease in weight during this test of not more than 0.10 percent after 7 days and 0.40 percent after 30 days. This permissible loss applies to the common plasticizers used in pvc in North America, but not to the fatty acid type sometimes used in Europe. These could be deleterious to freshly placed concrete.

The results of the various investigations and experience with its use in other applications support the belief that it will be more durable than ferrous metals or rubber.

WATERTIGHTNESS

The prime function of a waterstop is to prevent water from escaping through the construction or contraction joint across which it is installed. To determine the relative watertightness of different shapes of waterstops, comparative leakage tests were run on samples of all the types in Fig. 1.

Test arrangement

The tests were arranged so that various pressure heads could be applied, and so that the two sections of concrete supporting the waterstop could be separated to provide any desired joint opening. Fig. 2 and 3 show the general arrangement of the test setup and a cutaway section revealing the waterstop embedded in the concrete.

To prepare a sample for test, a length of waterstop was first formed into a ring by splicing together the two ends. With the labyrinth types a square "ring" had to be made by splicing four pieces together, since this type of material cannot be bent in the direction required in the test setup. Details of the splicing techniques will be described later. The ring was then suspended horizontally in the center of a box having a 2-ft square base and a height of 15 in. Concrete containing 1½ in. maximum size aggregate was placed around and inside the ring and brought level with the center line of the waterstop, leaving the upper half of the ring projecting above the concrete. After the first lift of concrete had hardened, pieces of building paper were cut to cover the whole of the "parting area" except for a circular hole in the center, large enough to accept an inverted metal pie plate (See Fig. 2). This pie plate had a hole cut in the center through which the water supply was fed. The top half of the box was then filled with concrete, completely enclosing the waterstop, but leaving a passage under the pie plate and the building paper for water to reach the waterstop.

Four bolts were set in the top and bottom sections of concrete to enable the two sections to be jacked apart for simulated joint movement. The separation of the top and bottom sections was limited by three pairs of tie rods which were also used for jacking the sections apart. Water from a storage tank was fed to the test rig through a piece of flexible rubber hose and pressure was applied to the tank from a compressed air supply equipped with a pressure regulator.

Leakage test

After a specimen of waterstop had been embedded, the concrete was cured until a strength of at least 3000 psi was attained in companion test cylinders. The water connections were then made and the two sections of concrete jacked apart about 1/32 in. This was done to insure a free water passage from the center of the block to the waterstop. The sections were then allowed to close again, and the tie rod nuts screwed down finger-tight. Water pressure was then applied at a pressure of 10 psi for 75 min. The first 15 min was a stabilizing period, during which any leakage was expected to reach a constant rate. During the remaining 60 min any water escaping past the waterstop was collected, and measured. From the volume of water collected and the length of waterstop used the rate of leakage per unit length of waterstop per unit of time was calculated. This process was repeated at pressures of 25 and 50 psi giving leakage rates corresponding to heads of 23, 58, and 116 ft of water. The initial tests were made with a closed joint. Subsequently, the tests were repeated with joint openings of ⅛ and ½ in. Finally, the joint was closed again and a further test run to establish whether watertightness is affected by previous opening of the joint.

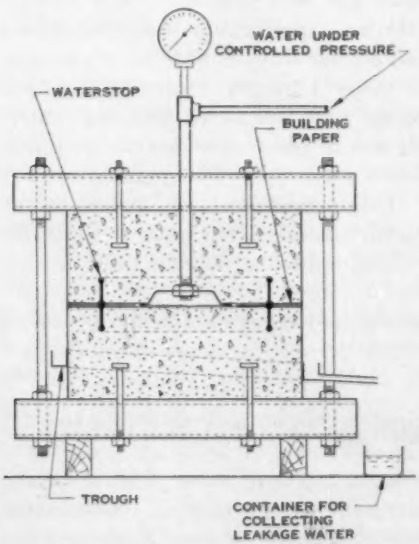

Fig. 2—Leakage test setup

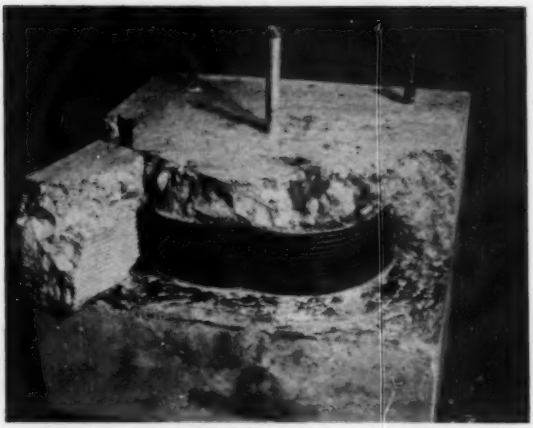

Fig. 3—Leakage test specimen

Results

The amount of leakage obtained, when considered in terms of gallons per foot per day, was in most cases negligible (Table 1). However, these tests were made under laboratory conditions with excellent consolidation of the concrete, and the results should be considered as comparative values rather than as the amount of leakage likely to occur on the job.

The types of waterstop showing the least leakage were the flat corrugated and the flexible metal types (Table 1). Of the four samples of flat corrugated waterstop tested only one showed any leakage and this was a small quantity past the heat-sealed splice in a 6-in wide waterstop at a joint opening of

TABLE 1—RESULTS OF LEAKAGE TESTS ON WATERSTOPS

Type	Size	Leakage, imperial gal. per linear ft per day									Remarks
		Joint closed			Open ⅛ in.			Open ½ in.		Reclosed	
		10 psi	25 psi	50 psi	10 psi	25 psi	50 psi	25 psi	50 psi	50 psi	
Labyrinth	2 rib	0	0	0	0	0	0	Pulled out		—	Believed to be from the four corner joints
	3 rib	0	0.035	0.021	0	0.133	0.222	0.645	0.756	—	
	4 rib	0	0	0.333	0	0	0.645	20.1	28.0	4.58	
Flat corrugated	5 in.	0	0	0	0	0	0	0	0	0	
	6 in.	0	0	0	0	0	0	0	0	0	
	6 in.	0	0	0	0	0	0	0.004	0.004	0	Clearly from joint
	9 in.	0	0	0	0	0	0	0	0	0	
Dumbbell 2-bulb	4 in.	0	0	0	0	0	0	0.389	0.401	0	
	6 in.	0	0	0	0	0	0	0	.022	0	
	6 in.	0	0	0	0	0.019	0.041	1.35	1.13	—	
	6 in.	0	0	0	0	0	0	0.160	0.187	T*	
	6 in.	0	0	0	0	0	0	0.167	0.401	0	
	9 in.	0	0	0	0	0	0	0.011	0.006	0	
	9 in.	0	0	0	0	0	0	0	0	0	
Dumbbell 3-bulb	9 in.	0	0	0	0	0	0	0	0	0	
	9 in.	0	0	0	T	T	T	T	T	0	
	9 in.	0	0	0	0	0	0	0	0	0	
Rigid steel plate	16 in.	78.8	—	—	—	—	—	—	—	—	
	16 in.	0	0	64.3	89.4	18.8	63.7	—	—	—	
Flexible metal	12 in.	0	0	0	0	0	0	0	0	0	

*T = trace.

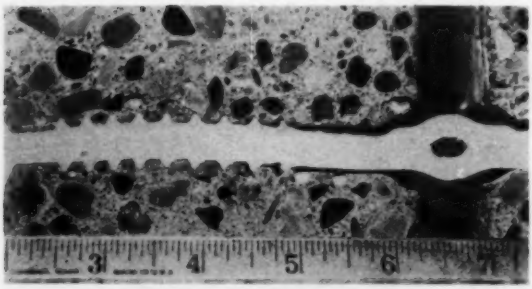

Fig. 4—A ¾-in. extension on a 9 in. flat corrugated waterstop, showing the anchorage provided by the ribs

½ in. The one sample of flexible metal waterstop that was tested showed no leakage.

The two larger sizes of labyrinth waterstop showed some leakage particularly when the joints were opened. It was evident from observations during the tests, however, that most of the leakage was from the four spliced corners. The fact that the smallest size, the two-rib type which was tested without corners in an earlier set-up, showed no leakage whatever, even with a ⅛-in joint opening, indicates that straight runs of labyrinth are not likely to leak. The labyrinth shape, however, is recommended only for construction joints where little or no movement is anticipated.

The dumbbell samples, while being watertight with closed joints, almost without exception leaked with a ½-in joint opening. The center bulb in the 3-bulb waterstop does not seem to add materially to watertightness for straight joint separation. In the 9-in dumbbell waterstop two samples of 2-bulb and three of 3-bulb were tested. One of each type leaked a small amount.

The rigid steel plate waterstops produced the most leakage of all types tested, regardless of whether the "sliding" half of the plate was coated with emulsified asphalt or a rubberized asphalt joint sealing compound.

To clarify the reason for the superior watertightness of the flat corrugated type over the dumbbell type in open joints, a short piece of each was embedded in concrete with the ends exposed to view and submitted to a tension test. When the dumbbell shape was extended the flat portion of the waterstop reduced in thickness, leaving only the bulbs at the ends in contact with the concrete. Since concrete is not completely impervious, some water can seep past this small area. When the corrugated section was extended (Fig. 4), the pull required to stretch the material was taken by the ribs nearest the center leaving the other ribs and the major part of the waterstop in intimate contact with the concrete.

MECHANICAL PROPERTIES

To be effective, a waterstop must have adequate strength to support the required head of water and must be sufficiently flexible to accommodate any movement occurring in the joint. The ability of waterstops to meet these

requirements has generally been established from hydrostatic tests. Results reported above indicate that a head of 110 ft with a ½-in joint opening is within the capabilities of even a 5 in. wide waterstop of relatively thin material. Ontario Hydro structures are normally supported on firm rock foundations. Movements between sections are therefore expected to be limited to those caused by thermal effects. Accordingly the maximum opening that can be accommodated by the larger sizes of waterstop was not established, nor was the effect of shearing movements.

Effect of temperature

In locations where one face of the waterstop is in contact with a large body of water, the temperature in winter can never drop very far. Where the moderating effect of water is not present during cold weather, waterstops may be subjected to the maximum joint opening at very low temperatures. Tests on sections of pvc waterstop representative of materials recently installed in Ontario Hydro projects indicated that joint openings of as much as 0.9 in. can be accommodated at a temperature of − 20 F.

Flexibility at low temperatures is also necessary to enable a waterstop to be manipulated during winter installation. Certain pvc waterstops have been found to be completely unworkable at a temperature of − 10 F. With proper compounding, however, a pvc can be made that will be adequately flexible at this temperature.

Toughness

A waterstop may receive rough handling on the job until after the concrete has been placed around it. The most critical condition for pvc is probably when half the width is exposed on the face of a section of concrete waiting the placing of the adjacent section. During this period the waterstop is exposed to the possible impact of heavy construction materials and equipment at temperatures which in northern Ontario are often as low as − 20 F. Toughness to withstand this treatment is therefore a necessary attribute. Experience to date indicates that both pvc and rubber waterstops are adequate in this respect. Toughness in a waterstop cannot be evaluated directly, but tests of tear resistance in conjunction with tensile strength and elongation at room temperature, and impact strength at low temperatures are believed to be reliable indicators. Pvc materials that have proved satisfactory in the past have exhibited tensile strengths of about 1800 psi, elongations of 280 percent, and tear resistances of at least 250 lb per in. Small sections of finished pvc waterstop of the same materials have resisted impacts of 40 ft-lb at − 20 F.

Stiffness

Another important property is the stiffness of the waterstop necessary to resist being folded during concrete placing. Experience to date is not too conclusive on the stiffness required. With careful placing it has been found on some jobs that a waterstop ½ in. thick at the center (excluding

TABLE 2—PHYSICAL PROPERTIES OF RUBBER AND POLYVINYL CHLORIDE WATERSTOP COMPOUNDS

Property	Number of compounds	Range	
		High	Low
Polyvinyl chloride compounds			
Tensile strength, psi	18	2142	1306
Elongation, percent	18	410	186
Tear resistance, lb per in.	15	431	188
Stiffness in flexure, psi	12	878	210
Rubber compounds			
Tensile strength, psi			
(a) Original	5	3270	2630
(b) After 48 hr oxygen bomb	5	2780	1690
Elongation, percent			
(a) Original	5	476	285
(b) After 48 hr oxygen bomb	5	492	356

hollow bulbs) made from a suitable pvc compound can be installed without being folded. On the other jobs difficulty in achieving this result has been overcome by lacing the protruding edge of the waterstop to a strategically positioned reinforcing bar.

Creep of polyvinyl chloride

One undesirable property of pvc is its tendency to creep under stress. This might lead to a waterstop slowly deforming under the hydrostatic load when the joint is at its maximum opening. Then on closure of the joint, the "bulge" might become pinched with the resultant possibility of fracture. The rate of creep increases with higher temperatures. To check the behavior of typical waterstop pvc's in this respect, a laboratory test was run on a piece of waterstop accepted for the St. Lawrence power project. A weight of 15 lb was applied to an area 0.3 in. wide and 1 in. long, representing a head of 104 ft of water. The test was made at the abnormally high temperature of 100 F to accelerate the effect. The results showed a decreasing rate of creep with time, a displacement of 0.035 in. being obtained after 2 months. It was therefore concluded that the amount of creep occurring under actual service conditions would be negligible.

Relative properties of rubber and pvc

As an indication of the relative properties of rubber and pvc compounds used in commercially available waterstops, the range of test results obtained in acceptance testing of waterstops for Ontario Hydro use during the past 5 years is summarized in Table 2. The tests for which results are reported for the pvc compounds are described in the Appendix on p. 1284, and those used for the rubber compounds in ASTM D 412-51T and D 572-53.

INSTALLATION

Besides being able to prevent the passage of water, a good waterstop should be easy to install. The installation of a waterstop necessitates numerous joints or splices between the intersecting pieces. Time consumed in making intersections is a major part of the total installation time. With metal waterstops, skilled labor is required to weld or braze the junctions, and once made, joints in thin metal waterstops are prone to damage before receiving the protection of the surrounding concrete. Portable electric machines for making butt splices, and factory-made tees, ells, and crosses are available for both rubber and pvc. These are simple to use and excellent splices can be made by semi-skilled labor. With pvc, reasonably neat and effective splices can be made using hand tools.

All, except the labryinth and a recently introduced split dumbbell type waterstop, require the formwork to be split to position the waterstop across the joint between two sections of concrete. The labyrinth and split dumbbell types are nailed directly to the face of the form, and thereby reduce the cost of installation.

For a waterstop to be watertight, it must be in intimate contact with the concrete in which it is embedded. The opinion has been expressed that the ribs on some designs are so fine and closely spaced that concrete would not form itself to the shape of the waterstop. Fig. 5 shows a concrete surface formed against a piece of labyrinth waterstop. The concrete was a lean, harsh mix with 3-in. maximum size aggregate. The surfaces formed clearly demonstrate that there is sufficient paste and aggregate of smaller sizes in this type of concrete to fill the grooves between the ribs. It can also be seen that the fine ribs, which are 1/16 in. wide and 1/32 in. deep, are faithfully reproduced in the concrete. However, it is only with systematic and thorough vibration that such intimate contact can be ensured over the entire length of waterstop. It is also desirable to restrict the use of labyrinth waterstops to vertical surfaces. When placed horizontally, some difficulty may be experienced in filling the grooves on the underside with concrete, and also in cleaning the grooves on the top side before placing the upper section of concrete.

SELECTION OF WATERSTOPS FOR ONTARIO HYDRO USE

Having gathered information on durability, watertightness, ease of installation, and mechanical properties, a decision had to be made on the most suitable types of waterstop to adopt for Ontario Hydro use. Maximum dur-

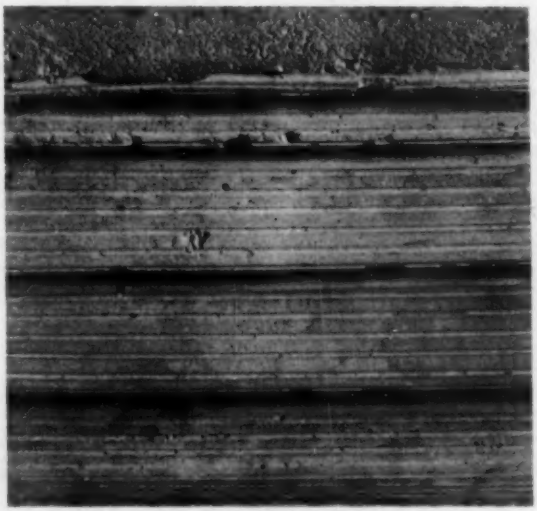

Fig. 5—Concrete surface formed against labyrinth waterstop

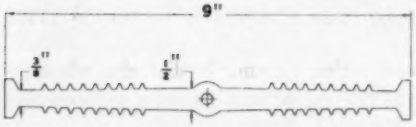

Fig. 6—Cross section of Type B waterstop developed for contraction joints

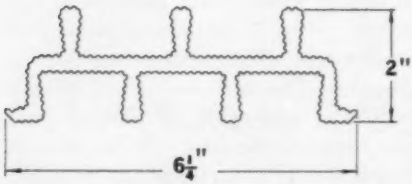

Fig. 7—Cross section of Type A labyrinth waterstop adopted for construction joints

ability and watertightness when installed under field conditions were the criteria considered to be most important.

As pointed out earlier the maximum assurance of durability should be obtained with stainless steel, copper, or a properly compounded polyvinyl chloride. So far as watertightness is concerned the laboratory tests showed the flat corrugated and the flexible metal types as performing equally well. It was felt, however, that with the flexible metal waterstops the difficulties of making joints and the possibility of damage before and during the placing of concrete around the waterstop might considerably reduce its ability to stop water. This reasoning pointed to flat corrugated waterstops made of pvc as being the most suitable combination. The investigational work also suggested that for construction joints where no movement is anticipated, the labyrinth shape, also made of pvc, should be adopted.

The outcome of these proposals was that for all contraction joints and some construction joints, a modified flat corrugated waterstop (Fig. 6) made of pvc was specified incorporating improvements suggested by the laboratory investigation. The longitudinal ribs provide the main barrier against the passage of water. The center bulb is incorporated to prevent tearing of the seal in the unlikely event of large lateral displacements, and the bulbs at the ends provide a last line of defense in areas where honeycombing results from poor placing of concrete. The thickness is believed to be such that, with the stiffness in flexure specified for the material, the waterstop will stand up against the pressure of the fresh concrete tending to fold it over during placing. For construction joints where the labyrinth shape provides savings in installation costs and where the orientation of the shape allows easy consolidation of concrete about it, a modified labyrinth waterstop, (Fig. 7), also of pvc, was specified having larger over-all dimensions and greater spacing between the ribs than found in commercially available shapes. These shapes were designated as Type B and Type A, respectively.

INSTALLATION TECHNIQUES—PVC WATERSTOPS

When the decision was made to adopt pvc waterstops for Ontario Hydro structures, a study was made of the techniques involved in installing the waterstops in the forms. Briefly, this operation consists of cutting the lengths required, splicing the pieces together, either in a shop or in the form,

and fixing the tailored waterstop in position. Various types of intersections are required at changes in direction and at junctions of one length of waterstop with another.

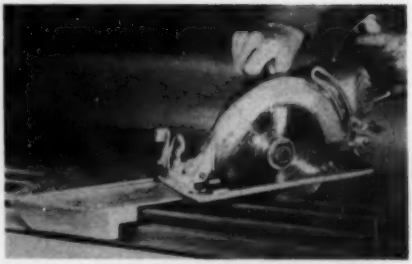

Fig. 8—Cutting labyrinth waterstop

Cutting waterstops

The method recommended by the manufacturers for cutting pvc waterstops was to use a sharp knife or hand saw. The knife was found to be better than the saw, but neither tool gave an accurate cut, and with heavy sections considerable effort was required. In the search for a quicker, more accurate method it was found that a carpenter's portable circular saw fitted with a hollow ground planer blade can make excellent cuts quickly (Fig. 8). The resulting clean cut makes splicing easier because of the close mating of the surfaces.

Splices

A splice in any flexible waterstop material must be strong and homogeneous. In the case of the ribbed designs, which depend on the continuity of the ribs for their ability to prevent the passage of water, it was found during the leakage tests that the ribs must be preserved in the area of the splice. Experiments using hand tools showed that an electrically heated soldering iron fitted with a variety of tips is convenient to use and produces satisfactory splices. The equipment used is shown in Fig. 9 and 10. The iron is of 300 watt capacity. The three knife-like tips are used for fusing the two surfaces to be joined and the larger grooved tip is for remolding the ribs on flat cor-

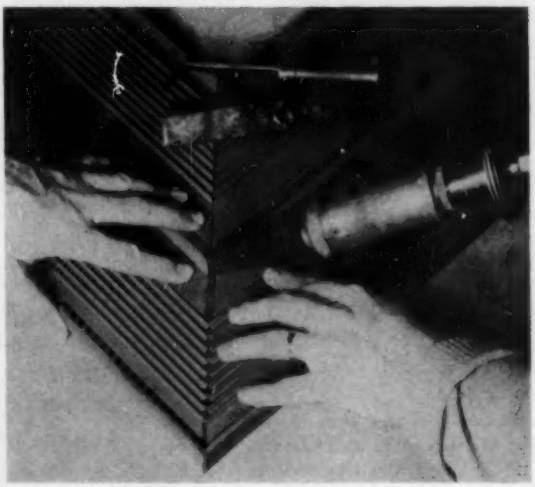

Fig. 9—Splicing a mitered corner in flat corrugated waterstop

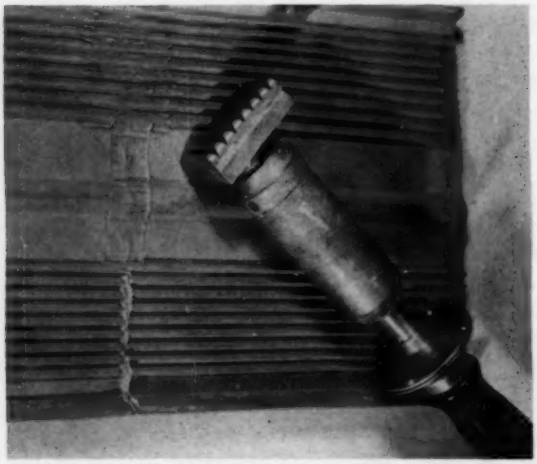

Fig. 10—Grooved tip in soldering iron for remolding the ribs on flat corrugated waterstop

rugated waterstops. For this purpose the grooves must match the ribs on the waterstop.

A considerable amount of experience is required before the more complicated intersections can be spliced by hand, and the results are seldom perfect. Greater assurance of a perfect splice can be had by using factory-made intersection pieces and a portable electric splicing machine, an experimental example of which is shown in Fig. 11. With such a device the labor time is reduced to a minimum. The ends to be joined are trimmed with the saw and inserted in the splicing machine. Heat for fusing the pvc is supplied by electric power controlled by a thermostat. Cooling is done by circulating cold water through internal passages in the machine. Test splices made in the laboratory with the machine illustrated proved to be very satisfactory.

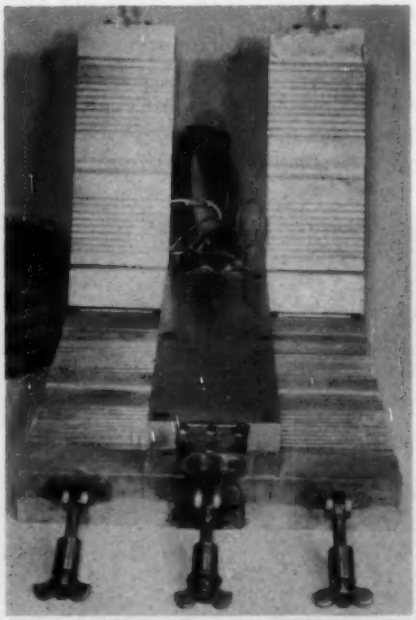

Photo courtesy Midland Enterprises, and Electrowert, Ltd.

Fig. 11—Splicing machine for flat corrugated waterstop

No matter what splicing method is used on the job, some system of inspection is necessary to ensure that all splices are properly made and that the waterstop is properly located in the form. Placing of concrete should not be allowed to start until the condition of the waterstops has been inspected and approved.

SPECIFICATIONS

A specification has been developed for use in the purchase of pvc waterstops for Ontario Hydro use.* In addition to specifying shapes and dimensional tolerances, it ensures adequate properties in respect of longevity, strength, elongation, toughness and stiffness, and cold weather flexibility. Most of these properties can be evaluated by ASTM test methods. New techniques have been developed for evaluating the other properties such as longevity and cold weather flexibility.

CONCLUSIONS

Durability—A properly compounded pvc is believed to be as durable as the best of the other materials in current use.

Watertightness—Waterstops, of the flat corrugated type, having anchoring ribs close to the center that relieve the outer parts of the waterstop from strain in the event of joint movement, and the flexible metal types, were the most watertight of those tested. The labyrinth type provided a watertight seal only when subjected to slight joint movements.

Mechanical Properties—Proper design and suitable materials can provide a waterstop that is easier to install than the metal types used in the past. Polyvinyl chloride materials can be compounded to have the mechanical properties required for installation even in the coldest weather experienced in Ontario, and also to have adequate strength and stability to perform reliably once installed.

ACKNOWLEDGMENT

The authors wish to acknowledge the assistance of J. C. Platt of the Commission's Research Division, many of the design engineers of the Engineering Division, and J. D. Blackburn, formerly of the Commission's staff.

APPENDIX

Excerpts from Technical Clauses of Typical Specification for Plastic Waterstops as Used by Ontario Hydro

Material

The waterstop shall be fabricated from a plastic compound, the basic resin of which shall be polyvinyl chloride. The compound shall contain any additional resins, plasticizers, inhibitors, or other materials such that, when the material is compounded, it shall meet the performance requirements given in this specification.

Single-pass reworked material of the same composition generated from the fabricator's waterstop production may be used. No reclaimed polyvinyl chloride shall be used.

All waterstops shall be molded or extruded in such a manner that any cross section will be dense, homogeneous, and free from porosity and other imperfections. The tolerances shown on the drawings shall govern all cross-sectional dimensions. The waterstops shall be symmetrical in shape.

*For excerpt from Ontario Hydro Designation C-245-55 see Appendix.

Qualification samples

The supplier shall furnish a sample of the material for each of the types of waterstop on which it tenders and which is in sheet form having a uniform thickness of from 1/16 to 1/8 in. and having a total area of not less than 2 sq ft. Each sample shall be comprised of pieces not smaller than 6 × 6 in. and shall be accompanied by an affidavit from the supplier to the effect that the sample is of the same material in all respects as that to be used in the manufacture of the finished waterstop.

In addition, the supplier shall furnish a 10-ft length of each type of waterstop he intends to supply.

Physical requirements

The waterstops or material from which the waterstops are fabricated shall meet the following requirements:

(*a*) *Tensile strength*—The tensile strength of the material shall not be less than: Type A, 1200 psi; Type B, 1400 psi.

(*b*) *Ultimate elongation*—The ultimate elongation of the material shall not be less than: Type A, 250 percent; Type B, 220 percent.

(*c*) *Tear resistance*—The tear resistance of the material shall not be less than: Type A, 175 lb per in.; Type B, 250 lb per in.

(*d*) *Stiffness in flexure*—The material shall have a minimum stiffness in flexure of: Type A, 200 psi; Type B, 350 psi.

(*e*) *Cold bend*—When tested as described below, cracking of the specimen where bent, as determined visually, shall be considered failure to pass the test.

(*f*) *Impact resistance*—When subjected to an impact of 40 ft-lb the material shall show no sign of cracking or chipping.

(*g*) *Accelerated extraction test*—The material, after having been subjected to the accelerated extraction test described below shall have a tensile strength of not less than: Type A, 1000 psi; Type B, 1200 psi; and an ultimate elongation of both types of not less than 200 percent.

(*h*) *Stability in "effect of alkalies test"*—The material, when tested in accordance with the effect of alkalies test described below shall have an increase in weight of not more than 0.25 percent at 7 days and not more than 0.40 percent at 30 days.

Material which exhibits a decrease in weight of more than 0.10 percent at 7 days or 0.30 percent at 30 days will be considered to have failed.

After 30 days immersion, the dimensions of the sample shall not differ from those of the original sample by more than 1 percent. After 7 days of immersion the Shore Durometer (Type A) reading of the sample shall not differ by more than ± 5 from the reading on the original sample.

Methods of test

Test specimens—Specimens for all tests, except as otherwise specified, shall be cut from sheets 1/16 to 1/8 in. thick supplied by the manufacturer. Specimens shall be of the shape and dimensions specified in the individual test methods.

Test conditions—Tests shall be conducted in a standard laboratory atmosphere of 70 to 75 F and 50 ± 2 percent relative humidity, unless otherwise specified in the testing methods or in these specifications.

(*a*) *Tensile strength and* (*b*) *ultimate elongation*—The tensile strength and ultimate elongation shall be carried out on both the sample as received and also on a sample which has been subjected to the accelerated extraction test described in (g) below in accordance with ASTM D 412-51T "Tentative Method of Tension Testing of Vulcanized Rubber," using Die *C*. The results shall be based and reported on the average of a minimum of five tests.

(*c*) *Tear resistance*—The "Standard Method of Test for Tear Resistance of Vulcanized Rubber" (ASTM D 624-48), using Die *B* shall be followed.

(*d*) *Stiffness in flexure*—A specimen 1 in. in width and 2 in. in length shall be fixed in a vise, having jaws of the smooth flat type, with one half of the length of the specimen free of the jaws. The vise shall be fixed to a suitable base and placed in a universal testing machine. The specimen shall be subjected to loading as a cantilever beam with the point of loading at a position ¼ in. from the face of the vise jaws. A loading head of greater width than the specimen and of suitable length shall be used to apply the load. The loading head should be of ⅛ in. material and shall be braced to prevent any flexure under load. The loading face of the head shall be ground to one side to form a knife edge and sufficiently rounded to prevent gripping of the specimen material by the head. The rate of loading shall be 0.2 in. per min. The stiffness in flexure shall be determined from the material load-deflection characteristics, using the following formula:

$$E = \frac{PL^3}{3\Delta I}$$

Where

E = stiffness in flexure, psi
P = applied load, lb
L = span length, 0.25 in.
Δ = deflection under applied load, in.
I = moment of inertia for specimen section, in.^{4}

The results shall be based on the average of at least three tests.

(*e*) *Cold bend*—A specimen of the material 1 in. wide and about 6 in. long shall be subjected to a temperature of − 20 F for 2 hr. On removal from the cooling chamber the specimen shall be immediately bent 180 deg around a rod of ¼ in. diameter with enough force to hold the specimen in intimate contact with the rod. The tests shall be carried out on at least three specimens and the results reported on each individual specimen.

(*f*) *Impact resistance*—A 4- to 6-in. length of the finished waterstop shall be fixed in a horizontal position in the jaws of a vise or by other suitable means. One half of the width of the material shall project free of the jaws. The vise shall be fixed on a base in such a manner so as to provide a vertical clearance of 5 in. between the specimen and the top surface of the base. The assembly shall be placed in a cooling chamber having forced circulation and subjected to a temperature of − 20 F for 3 hr.

Without removing the assembly from the cooling chamber, an 8-lb ball or weight shall then be dropped freely from a height of 5 ft so as to strike the projecting portion of the specimen squarely. The height of drop shall be measured from the top surface of the specimen to the bottom of the ball or weight. The tests shall be carried out at on least three specimens and the results reported on each individual specimen.

(*g*) *Accelerated extraction test*—Five tensile test specimens conforming to the shape and dimensions given in ASTM D 412-51T, using Die *C*, shall be cut from the sample submitted and the group weighed to the nearest milligram. The specimens shall be placed in a 1-l tall-form beaker with spout. The beaker shall be filled within 2 in. of the top of the beaker with a solution made by dissolving 5.0 g of chemically pure sodium hydroxide and 5.0 g of chemically pure potassium hydroxide in 1 l of distilled water. The specimens shall be completely immersed and the top of the beaker covered by a clock-glass. The beaker shall then be placed in a constant temperature bath which shall maintain the temperature of the solution between 140 and 150 F. A ¼ in. diameter glass tube shall be inserted into the spout of the beaker to within ½ in. of the bottom of the beaker. Air shall then be gently bubbled through the solution at the rate of about 1 bubble per sec. The solution shall be changed every 24 hr, the new solution being warmed to 150 F before replacing the old.

Once daily, each of the five specimens shall be removed from the beaker (preferably at the time of renewing the solution) and shall be rinsed lightly with distilled water. Each specimen shall then be superficially dried by means of a clean cloth. Ten min after the specimens have been thus dried the group of five specimens shall be weighed and the weight re-

corded. The sequence of testing shall be carried out continuously for not less than 14 days, after which, provided they have reached constant weight, they shall be tested for tensile strength and elongation. Under no circumstances, however, shall the specimens be tested for tensile strength until they have reached constant weight. Constant weight is assumed to have been obtained when three successive daily readings do not differ from one another by more than 0.05 percent of the original weight.

(*h*) *Test on the effect of alkalies*—A specimen, not more than ¼ in. thick weighing about 150 g, shall be cut from the sample supplied. Its dimensions to the nearest 0.001 in. and weight to the nearest milligram shall be recorded. The durometer reading, using the Shore Durometer (Type A), shall be taken in accordance with ASTM D 676-49T.

The specimen shall be completely immersed in a freshly made solution containing 5.0 g of chemically pure sodium hydroxide and 5.0 g of chemically pure potassium hydroxide per liter of distilled water, kept at 70 to 75 F. The solution shall be replaced by a similar fresh solution at a similar temperature every 7 days. At the end of 7 days the specimen shall be removed, rinsed with distilled water, and the surfaces wiped with a clean cloth. The dimensions, weight, and durometer hardness shall be measured and recorded. The specimen shall be reimmersed for 23 days, at the end of which time the dimensions, weight, and durometer hardness shall again be measured. At the age of 7 and 30 days any changes either in weight or dimensions shall be recorded as a percentage of the weight and dimensions of the original sample.

Nonstandard tests

The supplier is advised that certain additional tests which cannot be described as standard may also be carried out by the Commission on the finished waterstops. These particular tests are intended to further evaluate the cold weather handling properties and to assess the watertightness in concrete under various hydrostatic heads and at various joint openings. For example, one such test of flexibility of the waterstop under cold weather conditions is as follows:

A full width sample of waterstop 4 ft long and a 12 in. diameter mandrel shall be cooled in a refrigerator for a minimum of 3 hr at a temperature of − 10 F. After this period and while at this temperature, the material shall be sufficiently flexible to allow bending it *by hand* around the 12 in. diameter mandrel in not more than 4 sec.

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L-connectors and straight studs were tested in push-out specimens to get data on the fatigue behavior of stud shear connectors. Specimens were also tested under static loading. Based on these and previous tests, criteria are suggested for the design of composite beams with ½-in. L-connectors.

Fatigue and static strength of stud shear connectors

By BRUNO THÜRLIMANN

IN AN EARLIER PAPER, "Composite Beams with Stud Shear Connectors,"[1] the results of a laboratory investigation on a 32-ft span composite beam bridge were reported. After completion of these tests it was realized that additional information—particularly on the fatigue behavior of stud shear connectors—was highly desirable. For this reason a series of further tests were conducted.

TEST SPECIMENS

The specimens were of the push-out type. Such specimens have been used extensively in this country[2] and abroad[3] for determining the static strength of shear connectors. As shown in Fig. 1, a specimen consisted of an 8 WF 40 section framed by two concrete slabs. Four shear connectors were welded to each flange of the WF beam. Substantial reinforcement was placed in the slab, similar to the reinforcement of a deck slab in an actual composite beam bridge.

Three different types of specimens were used. Five specimens with ½-in. diameter L-connectors 2¼ in. high (Fig. 1, Detail A) had flat concrete slabs. Transverse reinforcing bars were placed into the bend of the L-connectors as indicated in Fig. 1. Straight studs (Fig. 1, Detail B) were used in three specimens of equal geometric proportions, the transverse reinforcement crossing in front of the studs. In the two haunched specimens with 3 in. deep haunches at 45 deg (Fig. 1) no connection existed between the L-connectors and the slab reinforcement. Table 1 lists the specimens and indicates the loading condition under which they were tested.

The specimens were fabricated as follows. The 8 WF 40 section was of A-7 steel. Stud material met the requirements of ASTM A 15-54T. The mechanical properties corresponded to the ones reported in Reference 1. Slab reinforcement was A 305 deformed bars. Both the L-connectors and stud connectors were "solid fluxed." No special preparation was given to the flange surface of the beam prior to welding. All specimens were cast at the same time in the same upright position as they were subsequently tested. Ready-mixed concrete with

TABLE 1—DESIGNATION OF SPECIMENS

Specimen No.	Slab	Stud				Loading
		Number	Type	Diameter, in.	Height, in.	
1	Flat	8	Straight	¾	4	Static
2	Haunched	8	L-connector	½	2¼	Static
3	Flat	8	L-connector	½	2¼	Static
4	Flat	8	L-connector	½	2¼	Cyclic
5	Flat	8	L-connector	½	2¼	Cyclic
6	Flat	8	L-connector	½	2¼	Cyclic
7	Flat	8	L-connector	½	2¼	Cyclic
8	Haunched	8	L-connector	½	2¼	Cyclic
9	Flat	8	Straight	¾	4	Cyclic
10	Flat	8	Straight	¾	4	Cyclic

a maximum aggregate size of 1 in. was used. Concrete was compacted with a mechanical vibrator. Concrete strength was checked on ten cylinders at appropriate intervals (Table 2).

FATIGUE TESTS

Fig. 2 illustrates the test set-up for fatigue loading. The testing frame was erected with standard parts of the fatigue testing installation at Fritz Engineering Laboratory.[4] The concrete slabs of a specimen rested on a ½ in. thick plywood panel to obtain a uniform distribution of the reactions from the floor. For the haunched specimens only the slab proper was supported on the panel, the haunch itself being free. A hydraulic jack with spherical seats at each end applied an axial load to the WF beam of the specimen. Cyclic loading at a frequency of 500 cycles per min was produced by an Amsler pulsator. During testing the maximum and minimum load levels were automatically kept constant at the preselected values. The hydraulic measuring device allowed the determination of the load within an accuracy of 2 percent.

Instrumentation consisted of two dial gages only, measuring the relative slip between the slabs and the steel beam at two diagonally opposite flange tips. A special device, shown in Fig. 3, was developed such that the gage recorded the maximum slip only. Piece (A), holding the dial gage, was mounted on a ½-in. diameter steel bar (B) imbedded in the concrete slab. The L-shaped piece (C) was held against piece (A) by two screws and spring washers. A flat bar (D), being welded to the steel beam, followed its movement during cyclic loading. As slip increased the L-piece (C) was progressively pushed down-

TABLE 2—STRENGTH OF CONCRETE

Cylinder No.	Age at test, days	Strength psi
1	6	2650
2	6	2970
		Average 2810
3	29	5270
4	29	5100
5	29	4600
6	29	5360
		Average 5080
7	41	6200
8	41	5760
9	41	5930
10	41	5780
		Average 5920

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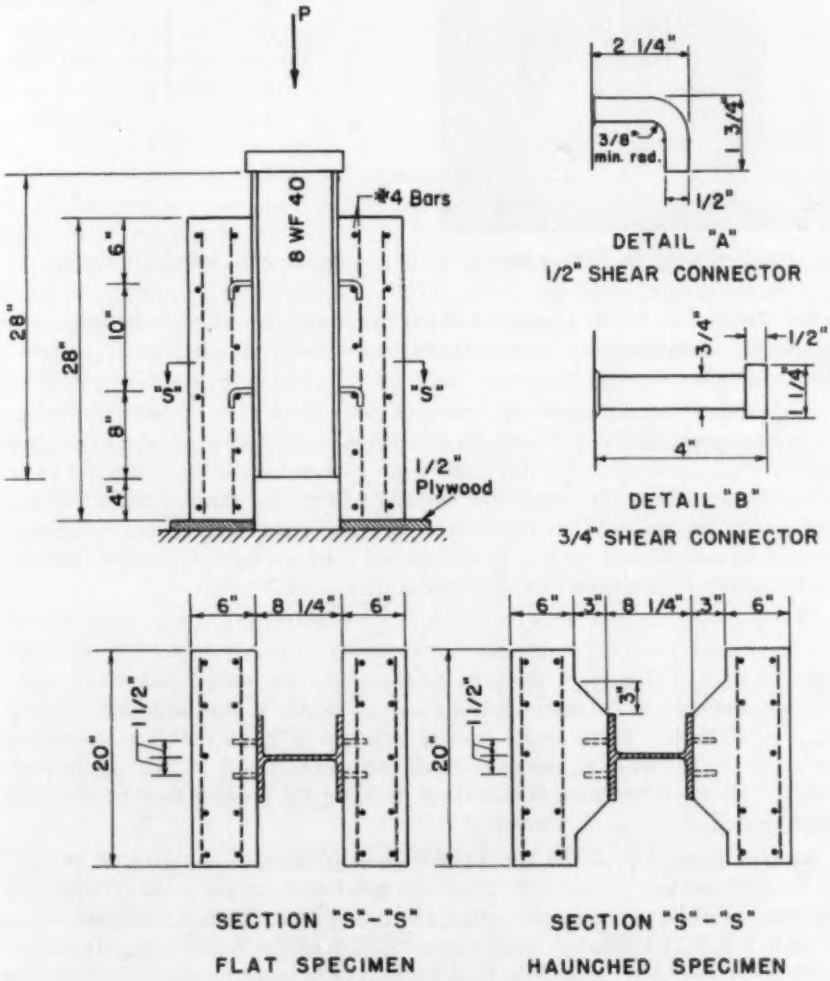

Fig. 1—Details of test specimens

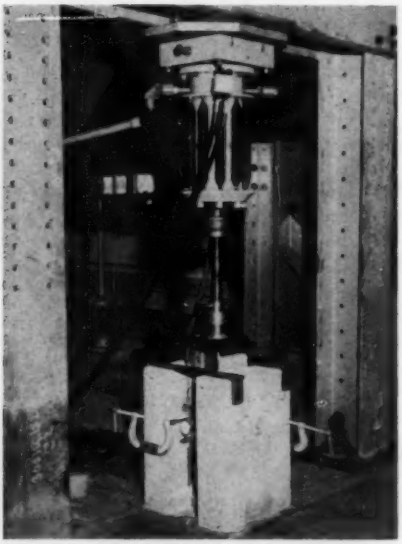

Fig. 2—Set-up for fatigue testing

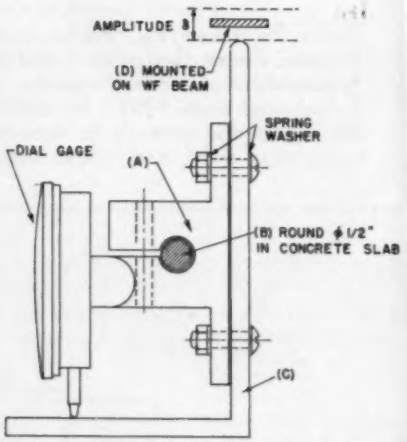

Fig. 3—Slip measuring device

ward such that the dial gage recorded the maximum amplitude only. The simplicity and reliability of the device was evident during actual operations and lead to satisfactory results.

No further instrumentation, and especially no strain measuring devices, were incorporated. It was felt that strain measurements on the studs were practically impossible. Furthermore, even if such measurements could be accomplished successfully, their use would be of rather limited value for it is sufficiently recognized that the fatigue and static strength of any connection cannot be established on the knowledge of a local peak stress but depends on the general conditions in the immediate vicinity.

Seven specimens, No. 4 to 10, Table 1, were tested in fatigue. The individual results are partially represented graphically in Fig. 4 to 8. For instance, Specimen No. 4 (Fig. 4) with ½-in. L-connectors was tested under load cycles varying between a maximum load P_{max} = 35,000 lb and a minimum load P_{min} = 4500 lb. These limits corresponded to average shearing stresses on the eight connectors of v_{st} (max) = 22,290 psi and v_{st} (min) = 2870 psi, respecttively. These values were obtained by dividing the applied load by the total cross-sectional area of all studs.

As seen from Fig. 4 the slip increased steadily with the number of load cycles and started to develop at an increased rate at about 150,000 cycles. Fracture of the studs occurred after 223,200 cycles. The slip measurements of dials I and II indicated almost equal slip over the entire test. However, a check of Fig. 5 to 8 reveals that in general a considerable difference developed in the two slip measurements, indicating that the connectors on one

side of the specimens experienced greater damage. Similar, but slightly less pronounced differences, developed in the static tests.

Test No. 8 on a haunched specimen (Fig. 6) deserves special mention. The initial loading producing a maximum average shearing stress v_{st} (max) = 13,370 psi and a minimum stress v_{st} (min) = 1910 psi, respectively, was kept constant over one million cycles. As no failure occurred and the average slip amounted to 0.00115 in. only, the load was increased in three consecutive steps. Failure finally took place at a stress v_{st} (max) = 22,290 psi and a total 2,815,000 cycles.

Under the testing conditions, illustrated in Fig. 1 and 2, fatigue cracks started from the lower side of the studs. In general a crack initiated at the reinforcement of the weld and developed a concave depression toward the beam, even removing some of the beam material. The failure surface of the studs showed a rounded shape. The typical crystalline texture of a fatigue fracture was apparent in the lower part. However, the upper region revealed marks of repeated impact, probably at the point of final fracture. The concrete surrounding the studs showed only local damage; around the base of the studs it was completely pulverized and could be blown out leaving a clearance of about 0.01 in.

Specimens No. 7 and 8 developed a fatigue fracture not only at the base of the studs but also approximately ¾ in. above the base. A possible failure mechanism is sketched in Fig. 9. The first fracture developed at the base of the stud. After considerable cracking the stud was still held in the concavity of the beam material. Under these conditions the bending moment

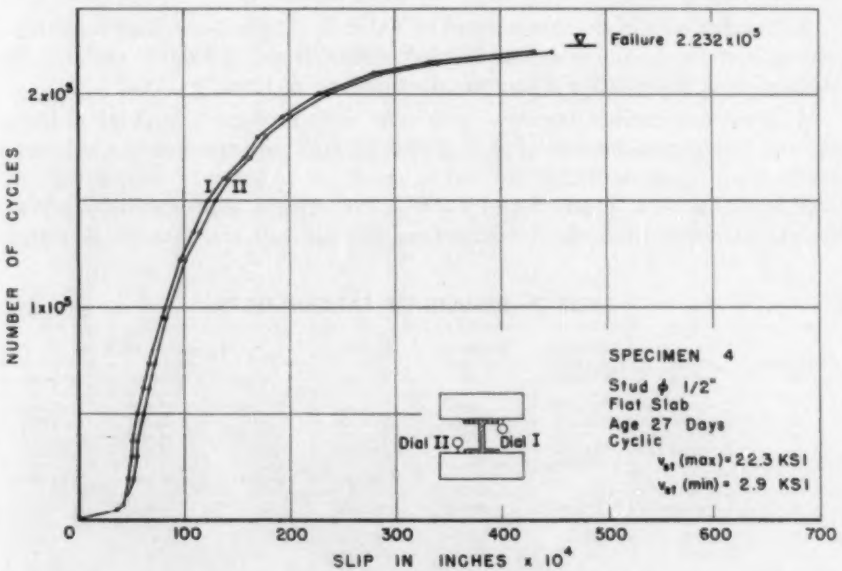

Fig. 4—Fatigue tests: Specimen No. 4

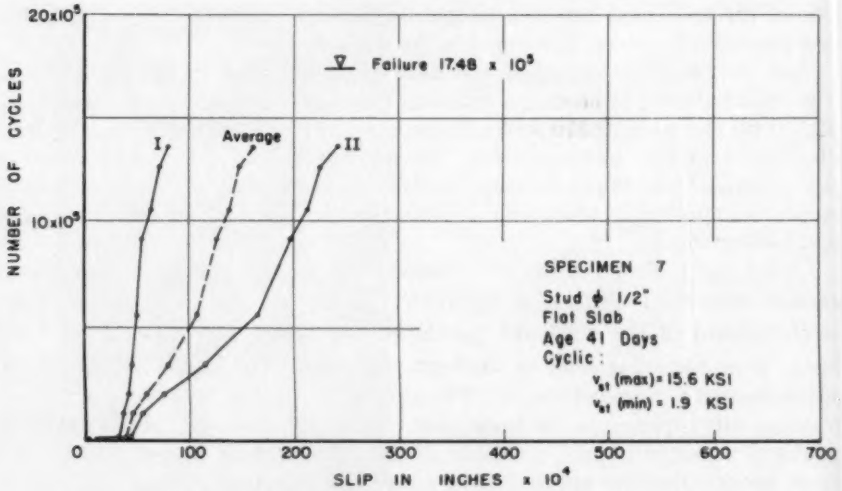

Fig. 5—Fatigue tests: Specimen No. 7

at the base of the stud was greatly reduced and a new maximum developed, as indicated in Fig. 9(B), causing ultimately the second fracture. It should be mentioned that both specimens, having L-connectors, were tested at relatively low stresses, sustained 1,748,000 and 2,815,000 cycles, respectively, and developed comparatively little slip prior to failure.

All fatigue results are summarized in Table 3. Table 3 also lists the fatigue strength of the L-connectors at 100,000; 600,000; and 2,000,000 cycles. The derivation of these latter values are discussed on p. 1296.

A direct comparison between specimens with L-connectors (Fig. 1, Detail A) and straight connectors (Fig. 1, Detail B) was performed at the two stress levels of v_{st} (max) = 22,290 psi and v_{st} (max) = 15,600 psi, respectively. As seen from Fig. 4, 5, 7, and 8 and Table 3, the straight studs exhibited a lower fatigue strength than the L-connectors, the ratio of the number of fatigue

TABLE 3—RESULTS OF FATIGUE TESTS

Specimen No.	Stud type	Maximum stress,* psi	Minimum stress,* psi	Cycles to failure	Fatigue strength, psi		
					$f_{100,000}$	$f_{600,000}$	$f_{2,000,000}$
4	L-connectors	22,300	2900	223,200	24,200	18,800	17,900
5	L-connectors	17,800	2200	134,200	18,300	15,400	13,600
6	L-connectors	17,800	2200	261,000	19,700	16,400	14,500
7	L-connectors	15,600	1900	1,748,000	20,500	17,300	15,400
				Average	20,700	17,000	15,400
8	L-connectors	22,300†	4800†	2,815,000	—	—	—
9	Straight	22,300	2800	169,400	—	—	—
10	Straight	15,600	1700	474,000	—	—	—

*Load divided by cross-sectional area of all studs.
†For final 825,000 cycles (compare Fig. 6).

cycles being 169,400/223,200 = 0.76 and 474,000/1,748,000 = 0.27 at the two stress levels, respectively. This behavior may be explained qualitatively by the following reasoning. The smaller ½-in. diameter L-connectors introduce smaller local stress concentrations in the concrete. It is generally recognized that the ability of concrete to sustain such loading conditions increases the more localized they are. Secondly, the bearing pressure of the studs on the concrete is about inversely proportional to the stud diameter. However, the stud force varies with the square of the diameter such that heavier studs produce higher bearing pressures. Finally, the hook of the L-connector produces a mechanical wedging action. The concrete, pressed against the connector, is prevented from escaping vertically by the hook. This results in an increased vertical pressure against the steel beam and consequently an increased friction. The upset head of a 4 in. high straight connector does not produce such action to a similar degree.

Comparing the behavior of the haunched specimen, No. 8, with ½-in. diameter L-connectors and the flat specimens carrying the same connectors the former showed at least equal if not superior performance (Fig. 5 and 6). It should be remembered that the L-connectors were only 2¼ in. high whereas the haunch was 3 in. deep. Furthermore, nc connection was provided between the slab reinforcement and the studs. At the final failure load P = 35,000 lb the average concrete shearing stress v_c in the haunch at an elevation equal to the height of the studs reached 58 psi. There was no indication of any cracking in the haunch of the concrete slab during the entire test.

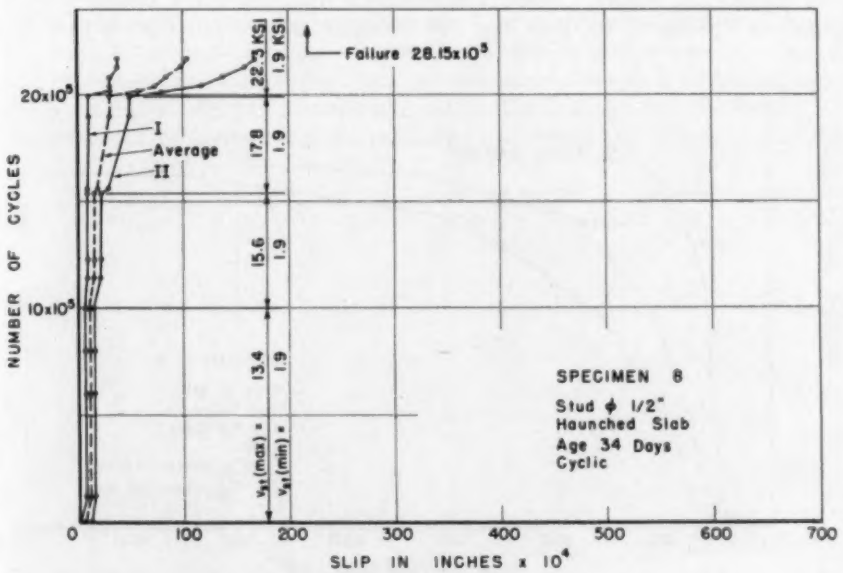

Fig. 6—Fatigue tests: Specimen No. 8

STATIC TESTS

One of each of the three types of specimens was tested under static loading. The specimen was placed in a standard hydraulic testing machine. Relative slip between the steel beam and the concrete slab was recorded by four dial gages located at the four flange tips of the WF beam. The loading was increased in steps. At appropriate levels the load was kept constant. After a waiting period to get the deformation stabilized, slip readings were taken. At high loads the waiting period increased to about 5 min.

Results of the three tests are plotted in Fig. 10 to 12, showing the average slip recorded by the four dials versus the average nominal shearing stress of the studs, the latter being the load divided by the cross-sectional area of all studs. For Specimens No. 1 and 2 the difference between the four individual slip readings did not amount to more than ±20 percent. However, for Specimen No. 3 it increased over 100 percent for high loads. It should therefore be recognized that not only under fatigue loading but also under static conditions a considerable spread must be expected between individual slip readings. Approaching ultimate load, the loading speed showed a marked influence on the value of the load itself. For this reason not too much significance should be attributed to the recorded ultimate load. Its value does not have any bearing on the design of shear connectors anyhow.

Fig. 12 also shows the end slip measured on the bridge specimen with ½-in. diameter L-connectors reported in Reference 1. The curve presents the average of the slips on the southeast and southwest end plotted in Fig. 8a and 8b of that report. The correspondence with the curve obtained from push-out Specimen No. 3 is fair. It therefore may be concluded that the

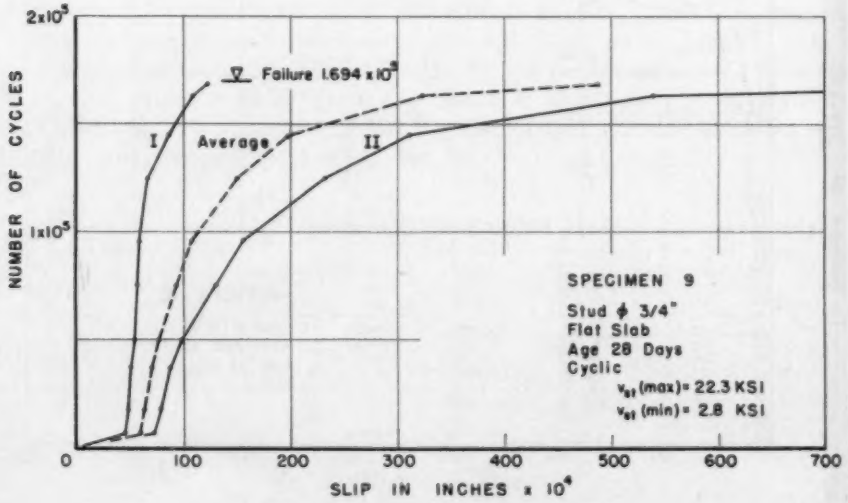

Fig. 7—Fatigue tests: Specimen No. 9

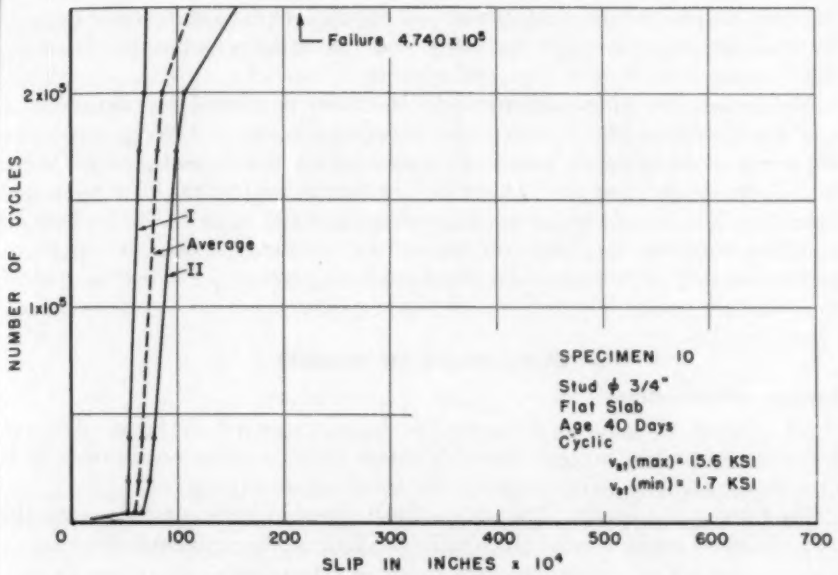

Fig. 8—Fatigue tests: Specimen No. 10

push-out test produced essentially the same conditions as normally exist in actual composite beams.

The relative behavior of ¾-in. diameter, 4 in. long straight studs and ½-in. diameter, 2¼in. high L-connectors can be judged by comparing Fig. 10 and 12. Specimen No. 3 with the ½-in. L-connectors showed a higher stiffness and less residual slip and hence confirmed similar findings.[1] A qualitative explanation can be given using the reasoning previously applied to explain the

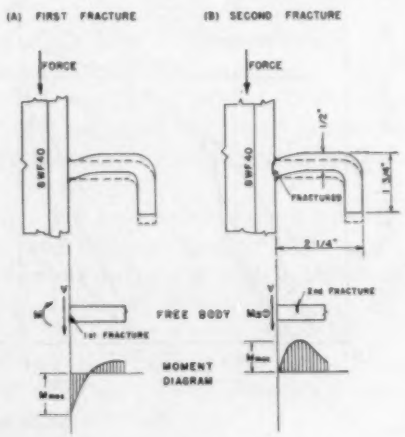

Fig. 9—Failure mechanism for studs of Specimen No. 8

superior fatigue behavior of these connectors. The smaller stress concentrations, the lower bearing pressures, and the wedging action produced by the L-connectors improve their performance.

No appreciable difference developed between haunched Specimen No. 2 and flat Specimen No. 3 with ½-in. L-connectors up to 0.05 in. slip. The difference in the ultimate load is of no significance as this load does not enter into design considerations. At the failure load P = 116,000 lb of haunched Specimen No. 2 the average concrete shearing stress v_c in the haunch at an elevation equal to the height of the connectors was 192 psi. It should be mentioned that in the test only the slabs were supported leaving the end of the haunches completely free.

EVALUATION OF RESULTS

Fatigue considerations

An attempt is made to delineate the fatigue strength of ½-in. diameter L-connectors and to suggest allowable values under working conditions which may be used as a basis of discussion for specification writing.

The fatigue test results obtained establish specific values of fatigue strength at preselected stress levels. To obtain the fatigue strength corresponding to a given number of cycles, recourse to some extrapolation procedure is necessary. Assuming the relationship

$$f = S\,(N/n)^k$$

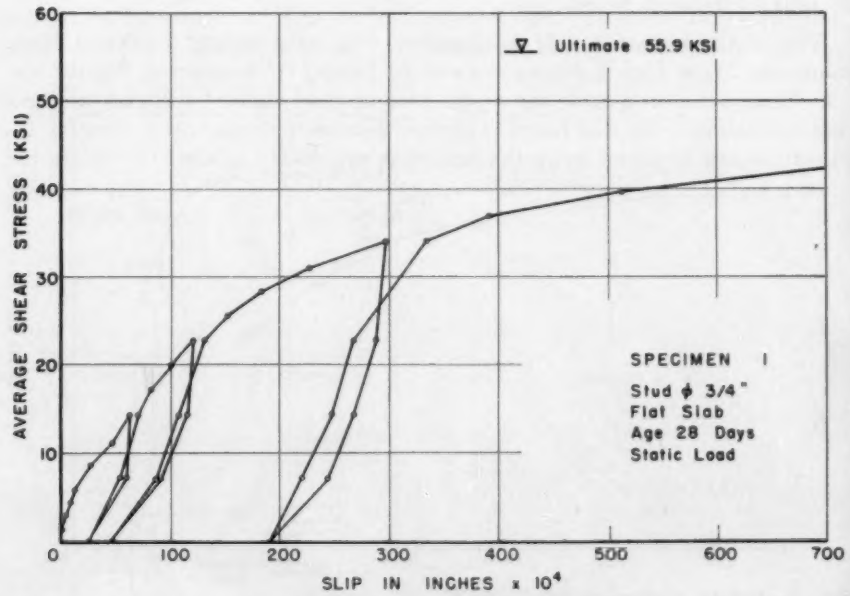

Fig. 10—Static tests: Specimen No. 1

applies,[5,6] where S is the stress at which a given specimen failed after N cycles, n is the number of cycles for which the fatigue strength f is desired, and k is an experimental constant, the value of f can be computed. The parameter k corresponds to the slope of a straight line which averages the individual test points in a logarithmic S-N diagram. From the results obtained on specimens with L-connectors and flat slab, Specimens No. 4 to 7, a value of $k = 0.10$ was established. This value is in the range of corresponding values obtained from fatigue tests of welded joints.[6] Using the above equation the fatigue strengths for 100,000; 600,000; and 2,000,000 cycles of Specimens No. 4 to 7 have been calculated and are listed in Table 3. The average values of the four results are as follows:

$$\begin{aligned} &\text{100,000 cycles:} && f_{100,000} = 20{,}700 \text{ psi} \\ &\text{600,000 cycles:} && f_{600,000} = 17{,}000 \text{ psi} \\ &\text{2,000,000 cycles:} && f_{2,000,000} = 15{,}400 \text{ psi} \end{aligned}$$

representing the fatigue strength of ½-in. diameter L-connectors for the corresponding number of cycles. The connector force producing failure is obtained by multiplying the cross-sectional area of the stud by the fatigue strength f. Remembering that in testing the minimum load amounted to about 12 percent of the maximum load, the above values are considered to be representative for zero to full load cycles also.

Allowable values under working conditions can be derived if the number of load cycles and the margin of safety are specified. No such criteria against the possibility of fatigue failure presently exist for any type shear connectors. However, as the connectors are welded to the steel beams it seems to be reasonable to follow the approach of "Standard Specifications for Welded Highway and Railway Bridges" of the American Welding Society[7] for the design of welded connections. Article 208 and Table I of that specification distinguish three types of loading for highway bridges depending on their length. The most severe conditions are produced by a "short critical loading (not more than two panels or 60 ft of loaded single lane)." The corresponding recommendations are based on an occurrence of 600,000 repetitions of full loading. The margin of safety incorporated into the allowable stresses depends on the type and orientation of the welds and is not constant. Appendix A of Reference 7 and reports of fatigue results[6] may be consulted for further information.

As composite beam bridges are generally used for shorter spans it was felt unnecessary to make any distinction between different loading cases in arriving at allowable values for shear connectors in fatigue. It is therefore proposed to use the most severe condition of 600,000 repetitions as a basis, Furthermore, on a comparative basis with the safety margins provided for welded joints, a safety factor $s = 1.25$ against fatigue is selected. Using these two criteria the allowable connector force for a ½-in. diameter L-connector under zero to full load cycles is

$$Q_{all} = \frac{A \times f_{600,000}}{s} = \frac{0.196 \times 17{,}000}{1.25} \leqq 2700 \text{ lb}$$

Where: $A = 0.196$ sq in. = cross-sectional area of stud

$f_{600,000} = 17{,}000$ psi = fatigue strength of L-connector for 600,000 load cycles

$s = 1.25$ = factor of safety against fatigue

The results obtained on the bridge specimen[1] give further support to the above value of Q_{all}. Under 1,000,000 cycles of design live load each stud was subjected to 2060 lb (see Table III, Reference 1); under a further 290,000 cycles of 125 percent live load the stud force was increased to 2580 lb. Even the final 250,000 cycles of 150 percent producing a connector force of 3090 lb did not result in any fatigue. It may therefore be concluded that $Q_{all} =$ 2700 lb presents a reasonable design value including a sufficient margin of safety against fatigue failure.

In actual bridges the shear acting on the connectors may not only vary between zero and a maximum value but reverse its direction. No tests under such alternating loading have been performed. However, it must be expected that such conditions will lower the fatigue strength of a connector to a similar degree as in welded joints. Following the approach used in "Standard Specifications for Welded Highway and Railway Bridges"[7] for welded joints, the allowable connector force Q_{all} could be made subject to the ratio maximum to minimum connector force.

No data are available concerning the influence of concrete strength on fatigue behavior of L-connectors. Average concrete strength of the push-out specimens at the age of testing (average age 32 days) reached about 5300 psi (Table 2). For the bridge specimen the corresponding values were 37 days and 3400 psi (Reference 1: Table I and appendix, Table B). The latter figures suggest that a minimum specified cylinder strength at 28 days, $f_c' =$ 3000 psi, is sufficient to insure the necessary fatigue strength.

Static considerations

Considering the useful static capacity of the L-connectors, Specimens No. 2 and 3 give additional support to the recommendations made in Reference 1. At a stress $v_{st} = 38{,}000$ psi the slip curve for Specimen No. 3 (Fig. 12) shows a definite break. The corresponding residual slip, as estimated from the figure, reached about 0.003 in. The slip curve of haunched Specimen No. 2 (Fig. 11) exhibits a somewhat less pronounced increase in slip at the same stress level. The increased residual slip of 0.007 in. should have no influence on a satisfactory performance. Multiplying the above stress by the cross-sectional area of an L-connector produces the connector force $Q =$ $38{,}000 \times 0.196 = 7450$ lb. This corresponds closely to the $Q = 7310$ lb determined on the bridge specimen as governing the useful capacity of L-connectors in accordance with the appropriate criteria on which the provisions for shear connectors of the AASHO specifications[8] are based.[9]

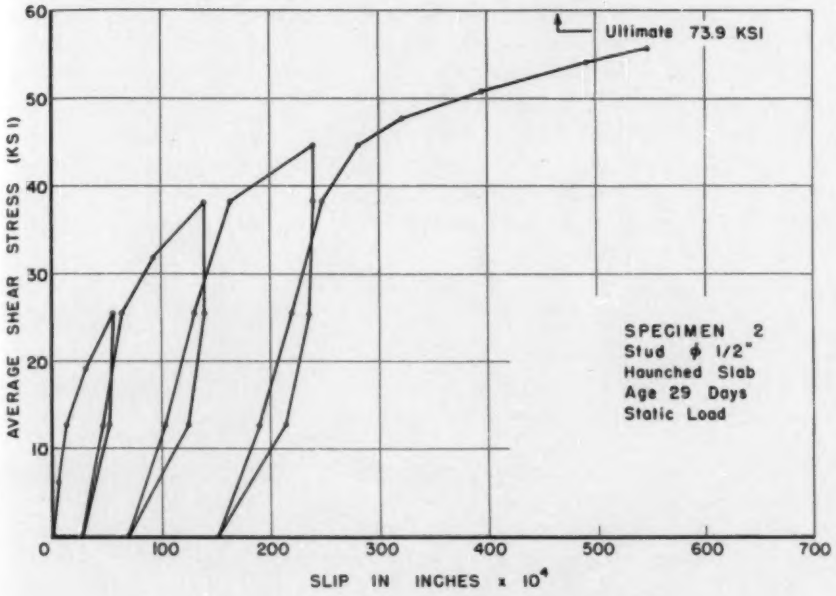

Fig. 11—Static tests: Specimen No. 2

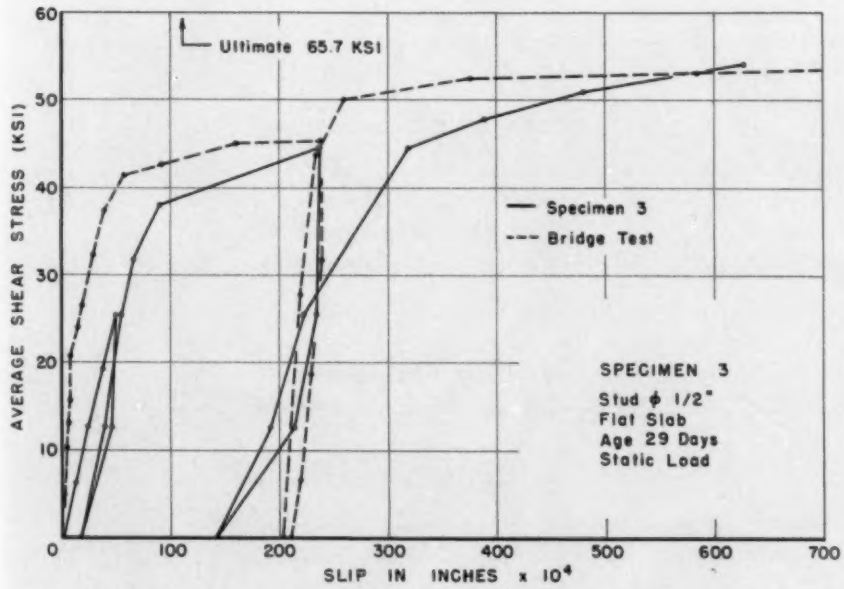

Fig. 12—Static tests: Specimen No. 3

In the tests reported here and in Reference 1 no explicit attempt was made to establish the dependence of the useful connector capacity on the 28-day cylinder strength, f_c', of the concrete. In Reference 2 an empirical relationship using $\sqrt{f_c'}$ has been proposed. The experimental evidence given is based on a strength variation of up to 550 psi only and shows considerable scatter (Reference 2, Table 4, p. 885). Reasoning that the influence of the concrete strength on straight studs and L-connectors should be similar, it was felt that for the time being a variation with $\sqrt{f_c'}$ should be used pending further experimental evidence.

Slab with haunches

The use of the relatively short 2¼ in. high L-connectors in conjunction with haunched slabs requires some special considerations. Depending on the shearing stresses present at a section through the top of the studs (2¼ in. above the top flange of the steel beam) shear reinforcement of the haunch may or may not be required. The haunched Specimen No. 2 withstood an average concrete shearing stress v_c = 192 psi at static ultimate load whereas Specimen No. 8 carried a stress v_c = 58 psi under fatigue at the failure of the studs. In the design of the haunches the appropriate requirements covering the web reinforcement of reinforced concrete members should be followed. If reinforcement is necessary it should be anchored to the L-connectors. In design, stresses produced by shrinkage and plastic flow are usually neglected. However, it is known that these effects may cause relatively large shearing forces at the ends of composite beams. A nominal amount of reinforcement of the haunches at the beam ends should be provided even if not required by the shearing stresses due to dead and/or live loads. It will prevent the development of possible shrinkage cracks along the haunch.

DESIGN RECOMMENDATIONS

Supplementing the recommendations of Reference 1 by the considerations presented in this paper, the following summary is presented for the design of composite beams with ½-in. L-connectors.

Bridge design

1. The geometric shape of the connector is shown in Detail A, Fig. 1.
2. The hook should preferably be oriented against the direction of the horizontal shear (toward middle for simple beams).
3. The useful static capacity of the shear connector in pounds is given by:

$$Q_{uc} = 120 \sqrt{f_c'}$$

Where f_c' = 28-day cylinder strength of concrete in psi.

The resistance value at working load is obtained by dividing Q_{uc} by the appropriate safety factor.*

*See Article 1.9.5, Reference 8.

4. To insure a sufficient margin of safety against fatigue the allowable maximum connector force, Q_{all}, produced by live load (or dead load plus live load in case of shoring of steel beams) should be limited to

$$Q_{all} \leqq 2700 \text{ lb}$$

The concrete at 28 days should have a minimum strength of 3000 psi.

5. If the slab is provided with haunches the shearing stress in the concrete at the height of the upper ends of the connectors should be checked. Reinforcement of the haunches, if necessary, should be provided in accordance with the applicable requirements for the web reinforcement of reinforced concrete beams.

Building design (primarily static loading)

1. The useful capacity of the shear connector in pounds is

$$Q_u = 120 \sqrt{f_c'}$$

2. The allowable maximum connector force under the specified working loads is

$$Q_{all} = \tfrac{1}{2} Q_u = 60 \sqrt{f_c'}$$

ACKNOWLEDGMENT

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Title No. 55-79

Effect of Powdered Minerals and Fine Aggregate on the Drying Shrinkage of Portland Cement Paste

KENNETH M. ALEXANDER and JOHN WARDLAW

■ THE EXTENT TO WHICH FINE AGGREGATE and powdered minerals of cement fineness affect the drying shrinkage of portland cement paste was studied under a wide range of conditions. The water and powdered admixture contents of the paste were varied both separately and simultaneously, and the effect of gradually increasing the size of the powdered mineral grains until the admixture became an aggregate was studied. Results show that if the portland cement in a paste is partially replaced by powdered basalt, of cement fineness, and at a constant water-(cement + admixture) ratio, the effect on drying shrinkage represents a balance between the individual contributions arising from the simultaneous changes in the water-cement and admixture-cement ratios. An equation is given which relates both shrinkage and strength changes to changes in the water and admixture contents.

When the effect of aggregate on drying shrinkage is considered in terms of the elastic constants of the embedded particle and the surrounding medium, some distinction should be made between the restraint imposed by rock particles small enough to fit between individual clinker grains, and that imposed by the same volume of rock in the form of grains or pebbles large enough to displace whole zones of clinker grains.

DRYING SHRINKAGE of portland cement paste depends on the proportion of mixing water used, and on the length of time during which water loss is restricted by curing. Also the magnitude of the shrinkage caused by subsequent drying is changed if the hardened mass contains not only cement and water, but particles of rock. As the proportion of rock changes, drying shrinkage also changes. Would shrinkage be modified to the same extent if the same proportion of rock could be introduced in the form of fine dust, at the same ratio of water to portland cement? What if the fine mineral powder admixture were pozzolanic?

Although any one of the factors mentioned can change ultimate drying shrinkage, an additional complication arises in practice since several factors tend to vary simultaneously. For instance, if cement is partially replaced by mineral powder, it is unlikely that the water content of the mix could be reduced. Under these conditions not only is cement displaced by small particles of rock, but the ratio of water to portland cement is increased simultaneously.

The present study of drying shrinkage was made for three reasons: first, to measure the extent to which shrinkage is modified when the factors defined above are changed one at a time, and under the same experimental conditions; second, to see if the change caused by altering more than one factor at a time is a simple combination of the way in which each factor, operating singly, would have influenced drying shrinkage; and finally, to find if a gradual or a

sudden change of behavior occurs between different factors. For example, when the particle size of fine aggregate used in a mortar is progressively reduced until the added material becomes virtually a mineral powder, the project which started as a study of "effect of aggregate" becomes a study of "effect of mineral powder."

Considerable background for the present investigation is provided by information published in connection with the drying shrinkage of straight portland cement pastes, mortars, and concretes. A representative selection of such work is reviewed in the *Symposium on Shrinkage and Cracking of Cementive Materials*,[1] published by the Society of Chemical Industry. Other general studies and reviews have been compiled by L'Hermite,[2] Giertz-Hedstrom,[3] and Hummel.[4] In addition, the effect of aggregate on drying shrinkage has been considered by Carlson[5] and Pickett.[6]

By comparison with the work on straight cement, relatively few studies have been made in which the drying shrinkage of cements blended with mineral powders forms the major part of the project. L'Hermite *et al.*[7] and Duntron[8] have given considerable attention to the question of the drying shrinkage of portland cement blended with inert mineral powders, while Davis *et al.*[9,10] have studied the drying shrinkage of cements blended with pozzolans.

MATERIALS AND METHOD OF TEST

Method

Drying shrinkage measurements were made on 1-in. square bars with 5-in. gage length. Specimens were mixed individually with a trowel, water being added from a burette. The air contents of the bars were found to be negligible. An initial curing period of 24 hr was completed in the molds which were stored in closely fitting air tight cans. Specimens were subsequently cured under water at 73 ± 2 F for 7, 28, 90, and 180 days. After removal from the curing bath the specimens were stored in a vacuum over silica gel at 73 ± 2 F. The bars were taken from the desiccator and measured with a dial gage comparator at intervals until shrinkage and weight loss ceased, usually after 3 to 5 months, although readings were continued up to 1 year. A minimum of four specimens was used for each series investigated. For the purpose of comparison extra specimens, which had been cured under water as described above, were subsequently allowed to dry in the air of a laboratory under conditions of varying humidity.

Materials

In all experiments with straight portland cement paste, or portland cement paste mixed with fine aggregate or basalt powder, a low alkali (Na_2O less than 0.6 percent) cement complying with the Australian Standard Specifications for Type C (low heat) cement was used. The hypothetical composition specified in Australia for Type C cement is given in Table 1, together with values calculated for the actual cement used.

Low alkali portland cement complying with the Australian Standard Specifications for Type A (ordinary or normal) cement was used in experiments on portland-pozzolan blends. The Type A cement had a surface area of 3280 sq cm per g, and the Type C cement an area of 3180 sq cm per g. The surface areas were determined with the Blaine air permeability apparatus at a porosity of 0.500 ± 0.005. The ultimate drying shrinkages of the two portland cements were identical, within limits of experimental error.

Fine aggregate and mineral powder were produced by crushing and ball milling coarse basalt aggregate from a deposit which regularly supplies aggregate for concrete work in the

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Melbourne area. The mineral powder was milled to a surface area of 3800 sq cm per g. Size gradings of the fine aggregate are given in the discussion section which follows.

The pozzolan (V17) was obtained by ball milling a reactive opaline aggregate from Butcher's Ridge, Gippsland, Victoria. This material, a milky-white to brown opal, has been used in previous studies on reactive aggregate[11] and pozzolanic reactivity[12]. Batches were ground to surface areas of 3800, 8500, and 85,000 sq cm per g, the latter two areas being measured by the air permeability method of Carmen and Malherbe.[13]

RESULTS AND DISCUSSION

The present investigation is essentially a study of the way in which the drying shrinkage of portland cement, cured under a range of conditions, is modified when part of the cement is replaced by various proportions of water and/or small particles of rock. The experimental work can be divided into three main subsections as follows:

1. Comparison between the present system of desiccation and uncontrolled air drying.
2. Investigation of the effect of a number of variables on the drying shrinkage of specimens previously cured under water for a fixed period. The factors studied include:
   a. Displacement of cement by water alone. Illustrated by Fig. 1a.
   b. Portland cement and water simultaneously replaced by basalt powder, at a constant water-cement ratio (Fig. 1b).
   c. Repeat of (b), using fine aggregate in place of basalt powder (Fig. 1c).
   d. Portland cement alone displaced by basalt powder at a constant water-(cement + admixture) ratio (Fig. 1d).
   e. Repeat of (d) using reactive pozzolan, milled to varying degrees of fineness, instead of "inert" mineral powder.

TABLE 1—COMPARISON OF CALCULATED COMPOSITION OF TYPE C CEMENT USED AND AUSTRALIAN STANDARD SPECIFICATION

Chemical compound	C_3A, maximum percent	C_4AF, maximum percent	C_3S, maximum percent	C_3S, minimum percent	SO_3, maximum percent
Australian Standard Specification A.2-1953 for Type C (low heat) portland cement	8	20	40	35	2.75
Calculated composition of Type C cement used in the present investigation	3	17	29	46	1.85

f. Transitional experiments between (b) and (c), in which the fine aggregate is progressively reduced in size until equivalent to mineral powder.

3. Study of the relationship between water loss and shrinkage for pastes subjected to different periods of water curing, and containing various proportions of either "inert" or pozzolanic mineral powders.

Comparison between experimental conditions and uncontrolled air drying

In Fig. 2 the drying shrinkages of straight and blended portland cement pastes desiccated under the present conditions of experiment are compared with those of identical specimens left for 1 year in the air of a room thermostatted at 73 F, but without humidity control, and in the presence of atmospheric carbon dioxide. In this diagram the drying shrinkages refer to bars previously cured under water for 28 days, and are plotted against the percent of cement replaced by powdered basalt, on a weight basis, at a constant ratio of water to cement + admixture (by weight). The two sets of conditions produce only slightly different drying shrinkages. A study of the way in which shrinkage progresses as drying proceeds, until values such as those recorded on Fig. 2 are attained, has been published by the authors.[14]

Dependence of drying shrinkage on paste and mortar composition

All drying shrinkages discussed in the six sections following apply to specimens cured initially under water for 28 days.

Displacement of cement by increasing the water content of the paste—When the water-cement ratio of straight portland cement paste was increased (Fig. 1a), the drying shrinkage increased linearly with the percentage of water present by volume (Fig. 3, Curve 1). Points on Curve 1 are for pastes with water-cement ratios of 0.15, 0.275, 0.36, and 0.45 by weight.

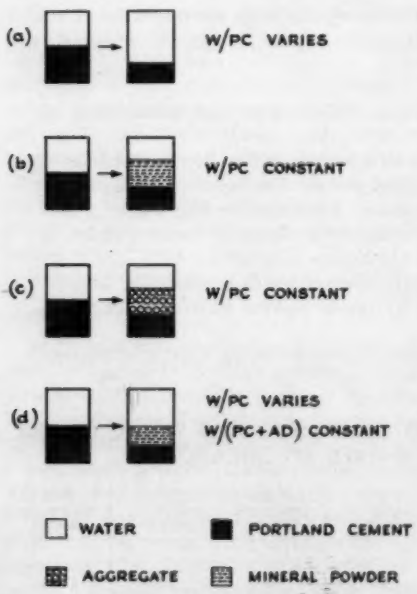

Fig. 1—Schematic representation of changes in mix proportioning

Simultaneous displacement of cement and water by powdered basalt—Cement and water were simultaneously replaced by basalt powder (Fig. 1b), producing blends with the same water-cement ratio, and different ratios of powdered basalt to portland cement. Since the ratio of water to cement + admixture decreased as the admixture content was increased, specimens containing higher proportions of powdered basalt were difficult to compact and were therefore prepared by mechanical tamping in the apparatus described by Alexander.[13]

Starting with a straight portland cement paste with a water-cement ratio of 0.45 by weight (Point A, Fig. 3), and replacing 20, 40, and 60 percent of the cement with an equal weight of powdered basalt, it was found that the shrinkages varied linearly (Fig. 3, Curve 2) with the sum of the volumes of water and powdered mineral in the paste. The higher the proportion of basalt present, the lower the shrinkage. This process of "diluting" the cement paste with rock particles is analogous to increasing the aggregate content of a rich mortar, at constant water-cement ratio, a procedure which also diminishes drying shrinkage. Curve 2 of Fig. 3 suggests that the increased shrinkage often associated with the use of nonreactive powders in cement is caused by the increased ratio of water to portland cement, since a reduction in shrinkage is observed when the water-cement ratio is held constant during the introduction of basalt powder. Additional comment on this point is given later.

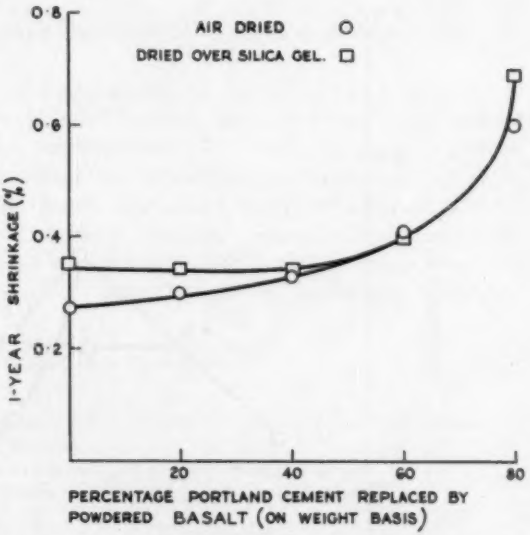

Fig. 2—Comparison between experimental conditions and uncontrolled air drying

Simultaneous displacement of cement and water by fine aggregate—The process of displacing cement and water simultaneously with fine aggregate, and at constant water-cement ratio (Fig. 1c), is analogous to the situation just described. The well known and main practical difference was that mixes containing a given proportion of fine aggregate were more easily compacted than the corresponding mixes with the same water-cement ratio, and with the same proportion of rock particles added in the form of fine powder.

Starting as before with a straight portland cement paste at the water-cement ratio of 0.45 by weight (Fig. 3, Point A), and replacing 20, 40, and 60 percent (by weight) of the cement with fine aggregate, a progressive decrease in shrinkage was observed (Fig. 3, Curve 3). At each replacement level the shrinkage reduction was greater than that caused by introducing an equivalent amount of the same rock in the form of mineral powder. In these experiments the fine aggregate consisted of particles within the restricted size range of − 20 + 30 mesh (U. S. standard sieve sizes).

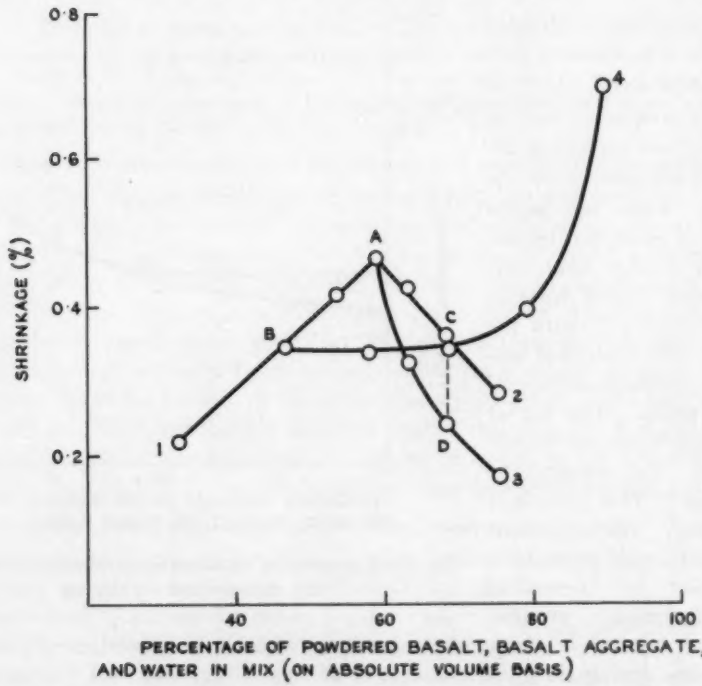

Fig. 3—Dependence of drying shrinkage on water, powdered mineral, and aggregate content of pastes and mortars cured under water for 28 days

Curve 1—Straight portland cement pastes with water-cement ratios of 0.15 to 0.45 by weight
Curve 2—Pastes containing 0 to 60 percent (by weight of dry cement) of basalt ground to cement fineness. Water-cement ratio constant at 0.45 by weight.
Curve 3—Mortars containing 0 to 60 percent, by weight of − 20 + 30 mesh basalt aggregate. Water-cement ratio 0.45 by weight
Curve 4—Pastes containing 0 to 80 percent, by weight, of basalt ground to cement fineness. Varying water-cement ratio; ratio of water to cement + admixture constant at 0.275 by weight

Displacement of portland cement by mineral powder at a constant water-(cement + admixture) ratio—The displacement of portland cement alone, by powdered basalt, and at a constant ratio of water to cement + admixture (Fig. 1d) approximates the conditions under which pozzolans and other mineral powders are used in practice. As the proportion of cement replaced by mineral is raised the water-cement ratio increases simultaneously. However, the tendency for shrinkage to increase as water-cement ratio increases is counteracted by the stabilizing effect of the particles of powdered mineral. With the basalt powder used here, and starting with a straight portland cement paste at a water-cement ratio of 0.275 by weight, the resultant effect (Fig. 3, Curve 4) was an unchanged or slightly reduced shrinkage when up to 40 percent by weight of the cement was replaced by admixture. At higher replacement levels the effect of the rapidly increasing water-cement ratio predominated, and caused greatly increased drying shrinkage. It is to be

expected that with wetter mixes the increase in shrinkage would begin at lower replacement levels.

Since the resultant effect of replacement at constant water-(cement + admixture) ratio is due to the balance between the individual contributions of the water and basalt powder, the mechanism by which shrinkage is altered is basically the same as that causing the accompanying changes in strength. For example, Alexander[16] has shown that when an inert mineral powder (limestone) of cement fineness replaces portland cement at approximately constant ratio of water to cement + admixture, the strength change is related to the change in the water content, and the level of replacement, by the expression:

$$\log \Delta S_{0-R} = \log \Delta x_{0-R} + aR + b \quad \ldots\ldots (1)$$

where

ΔS_{0-R} = strength change when powdered limestone content of portland cement increased from 0 to R percent
Δx = accompanying change in the ratio of water to portland cement
R = proportion of portland cement replaced by powdered limestone
a and b = constants

Within the limits of experimental error this same type of equation also applies to the accompanying changes in drying shrinkage when any powdered mineral admixture is used. Replacing strength change ΔS by shrinkage change $\Delta S'$, Eq. (1) can be written:

$$\log \left(\frac{\Delta S'_{0-R}}{\Delta x_{0-R}} \right) = a' R + b' \quad \ldots\ldots (2)$$

where a' and b' are constants.

Application of Eq. (2) to cement pastes containing admixtures which have been ground to cement fineness is shown in Fig. 4, where the examples given are:

a. Powdered basalt-cement blends at a ratio of water to cement + basalt = 0.275 by weight (from Curve 4, Fig. 3).
b. Pozzolan V17-cement blends at a ratio of water to cement + pozzolan = 0.35 by weight. (Data from Curve 1, Fig. 5).

Displacement of portland cement by pozzolan at a constant water-(cement + pozzolan) ratio—This situation is equivalent to that described in the preceding section and illustrated by Fig. 1d, with the difference that the powdered admixture was pozzolanic, and that the experiments were repeated with powders of more than one fineness. The ratio of water to cement + admixture was raised to 0.35 by weight since the equipment could not compact blends containing high proportions of the 85,000 sq cm per g pozzolan at lower water-cement ratios.

After a 28-day period of water curing, Pozzolan V17 ground to cement fineness (Fig. 5, Curve 1) affected drying shrinkage in much the same way as basalt powder of similar fineness (cf. Fig. 3, Curve 4). However, significant departures from this mode of behavior were apparent when the

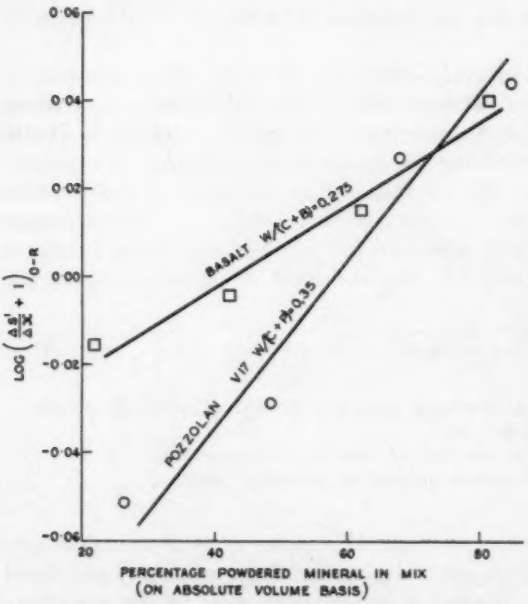

Fig. 4—Relationship between proportion of cement replaced by powdered admixture, and accompanying changes in drying shrinkage and water content. Values shown are by absolute volume, while $\Delta s'/\Delta x$ has been kept positive by displacing the R-axis one unit

fineness of the pozzolan was increased. The first major change to be noted at 8500 sq cm per g occurred at the replacement level of 60 percent by weight, where the upward trend of the shrinkage curve reversed (Fig. 5, Curve 2). On increasing the fineness to 85,000 sq cm per g the reversal moved to the lower replacement level of 40 percent (Fig. 5, Curve 3), and took place before replacement led to any large increase in shrinkage. In addition, with this finely ground material, the initial range (0–40 percent) over which shrinkage changed little with replacement, was absent.

As stated in connection with powdered basalt, it is to be expected that increased drying shrinkage would be encountered at lower replacement levels when higher water-cement ratios than those used here are employed with pozzolans which are sufficiently fine to comply with practical requirements. However, general conclusions relating to the effect of pozzolans on drying shrinkage cannot be drawn from the evidence presented here owing to the broad range of behavior patterns made possible by the widely differing reactivities and other properties of pozzolans.

Transition in behavior in changing from powdered mineral to fine aggregate—Starting with straight portland cement paste, and replacing 40 percent by weight of the cement with particles of basalt at a constant water-cement ratio, a drying shrinkage of approximately 0.35 percent was obtained when basalt of cement fineness was used (Fig. 3, Point C). Under otherwise identical conditions, the same proportions of basalt in the fineness range of − 20 + 30 mesh (U. S. standard sieves) reduced the shrinkage to approximately 0.25 percent (Fig. 3, Point D). The transition between Curves 2 and 3 was followed by studying mortars in which the fine aggregate fell within one of the following size ranges: − 20 + 30, − 50 + 100, − 200 + 270

mesh; 3800 sq cm per g. In Fig. 6 the drying shrinkages of these mortars are plotted against the mean particle diameter of the "aggregate" and show a relationship which is linear, within the accuracy of the experiment.

Pickett[6] has shown that shrinkage S in the presence of aggregate, is related to the shrinkage S_0 that would occur if no aggregate were present, by the equation:

$$\log\left(\frac{S_0}{S}\right) = \alpha \log\left(\frac{1}{1-g}\right) \quad \ldots (3)$$

where

g = volume of aggregate per unit volume of mix

α = a constant depending on the elastic constants of the aggregate particle and the surrounding medium

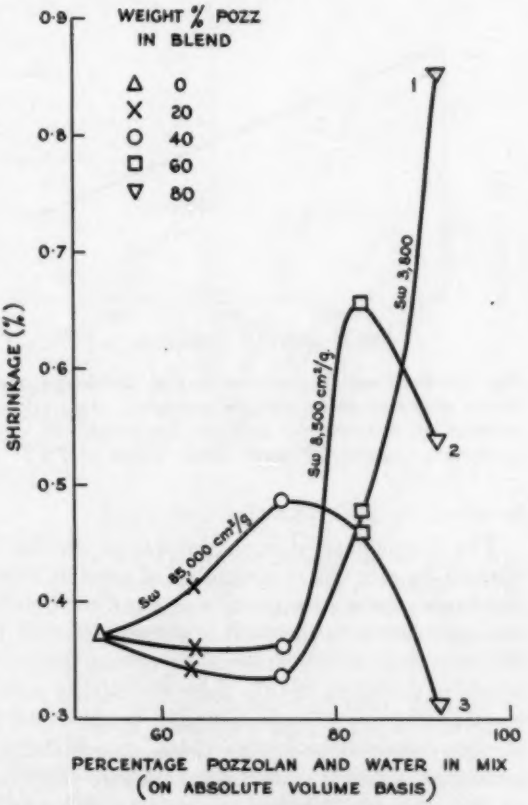

Fig. 5—Dependence of drying shrinkage on proportion of cement replaced by pozzolan of different degrees of fineness. Ratio of water to cement + pozzolan = 0.35 by weight. Duration of water curing, 28 days

From experiments on specimens cured 7 days under water before drying, Pickett found a value of $\alpha = 1.7$ for water-cement ratios of 0.35 and 0.50 by weight. If Eq. (3) is applied to the present mortars containing − 20 + 30 mesh aggregate (Fig. 3, Curve 3), a value of α of approximately 2.4 is obtained (Fig. 7, Curve 1). Similarly, applying Eq. (3) to "mortars" containing "aggregate" of cement fineness, and no coarser sand particles, an α of approximately 1.0 is obtained (Fig. 7, Curve 3). The mean of these two values would represent a crudely graded sand containing both coarse and fine particles. This latter value (1.7) is in agreement with that determined by Pickett.

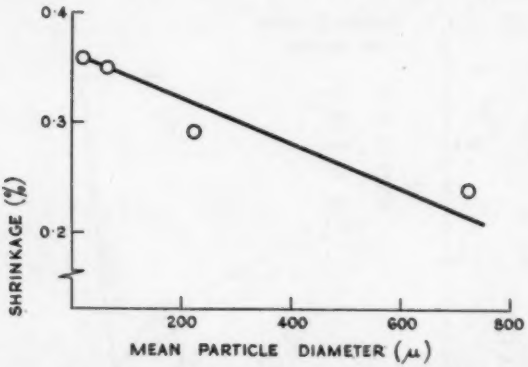

Fig. 6—Relationship between drying shrinkage and mean diameter of aggregate particles. Aggregate content of mortars: 40 percent by weight of dry cement. Cured 28 days under water at 73 F

It appears therefore that there is some difference between the restraint imposed by rock particles small enough to produce a blended cement paste by fitting between individual clinker grains, and that imposed by the same volume of rock in the form of grains or pebbles which are large enough to displace whole zones of clinker grains, while at the same time leaving the physical composition of the paste between the rock particles unchanged.

The dependence of α on the size of the fine aggregate grains may be explained by any one of a number of possible mechanisms by which the elastic constants of the rock particle and the surrounding medium could apparently change relative to each other when the size of the restraining particle alters. For example, tensile stresses at certain points in a hardened cement paste which is shrinking onto a large restraining pebble will be greater than those encountered anywhere in a similar paste which contains the same proportion of rock dispersed as fine particles. Since the tendency of the paste to creep and crack would be different in the two cases, the compressive stress exerted on the embedded aggregate would be different. The resultant effect is that a skeleton of larger aggregate particles is more effective in inducing volume stability when the paste dries. However, as the paste hardens with age, greater compressive stresses are exerted on even the smallest restraining particles, which therefore become more effective than previously in stabilizing the mass. Thus α for pastes containing basalt of cement fineness increases from 0.7 for specimens cured for 7 days under water (Fig. 7, Curve 4), to 1.0 (Curve 3) and 1.3 (Curve 2) for 28-day and 90-day curing periods, respectively.

Effect of curing time

In all experiments discussed to this point a fixed curing period of 28 days under water was used. The effect of increasing or decreasing the duration of water curing will now be considered, using data obtained with straight portland cement paste, and blended pastes containing 20, 40, and 60 percent of powdered basalt by weight of cement at a constant water-cement ratio of 0.45 by weight. In Fig. 8 the drying shrinkages attained by these specimens are plotted against the length of time cured under water.

It appears from Fig. 8 that the increased shrinkage caused by extending curing beyond 28 days is relatively small. In connection with the 28-day curing period it has already been stated (p. 1306) that simultaneous replacement of cement and water by powdered basalt, at a constant water-cement ratio, gives rise to a linear relationship between drying shrinkage, and the volume of water + powdered basalt in the paste. If the data from Fig. 8 are rearranged it will be seen that, within the limits of experimental error, linear relationships are obtained for all curing periods studied here (Fig. 9), and that the slopes of the lines depend on curing time.

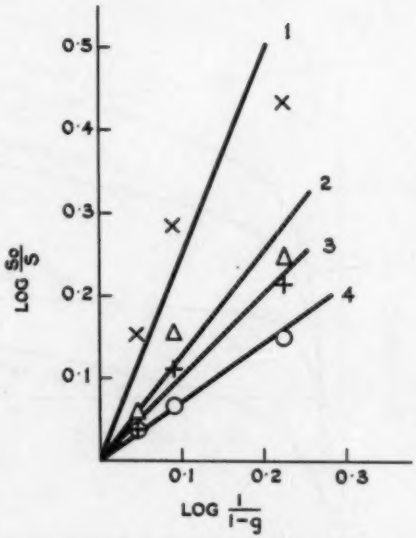

Fig. 7—Effect of aggregate grain size and duration of curing period on drying shrinkage characteristics of pastes and mortars. Curve 1—Mortars containing − 20 + 30 mesh basalt aggregate; cured 28 days under water. Curve 2—"Mortars" containing basalt "aggregate" of cement fineness; cured 90 days. Curves 3 and 4—Same as Curve 2, but with curing periods of 28 and 7 days, respectively

If all the cement and water could be replaced by mineral the result would be a solid block of dry rock showing no shrinkage on exposure to dry air. The behavior of such a material would be represented by Point A in Fig. 9. It appears that a reasonable series of straight lines can be drawn through Point A and through the experimentally determined values for shrinkage at different replacement levels. The slopes of these lines depend on the duration of the curing period in the manner illustrated by Fig. 10, and, as in the case of α (Fig. 7), slope changes reflect differences in the restraining influence that fine mineral particles exert on hydrated cement blends which have been cured for different periods under water.

CONCLUSIONS

1. When the water-cement ratio of portland cement paste is increased, the ultimate drying shrinkage increases linearly with the percentage of water present by volume.

2. If the portland cement in a paste is partially replaced by "inert" mineral powder such as powdered basalt, of cement fineness, and at a constant ratio of water to cement + admixture, the resultant effect on drying shrinkage represents a balance between the individual contributions arising from the simultaneous changes in the water-cement and admixture-cement ratios, and is given by the equation:

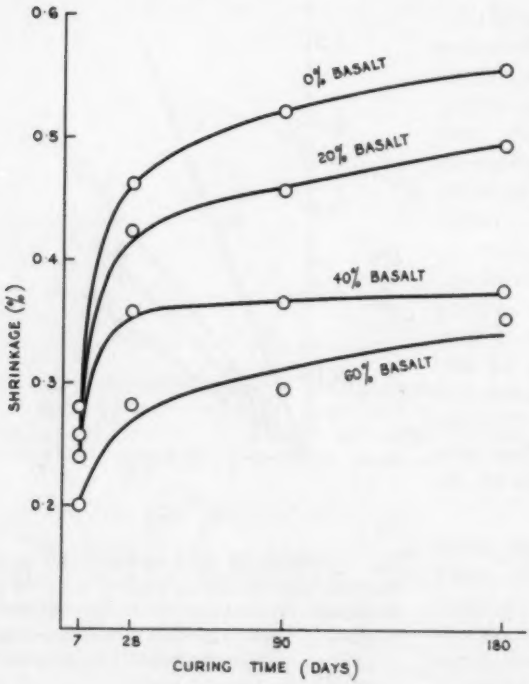

Fig. 8—Dependence of drying shrinkage of straight and blended portland cement pastes on duration of water curing. Percentage, by weight, of portland cement replaced by basalt of cement fineness as indicated on curves

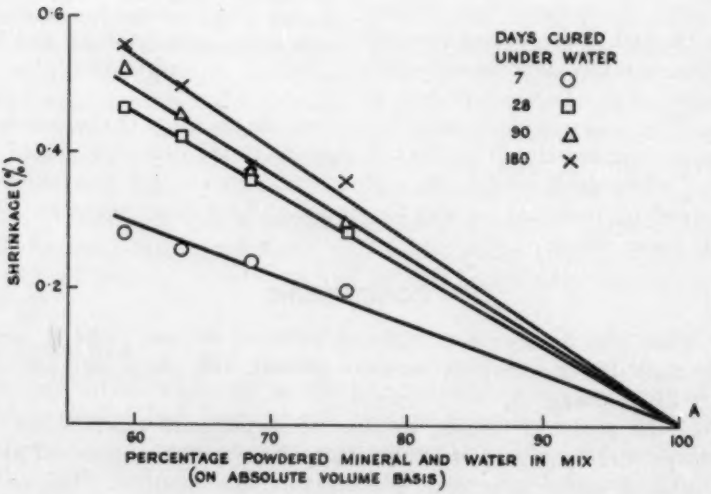

Fig. 9—Effect of curing time on drying shrinkage of portland cement paste containing 0 to 60 percent, by weight, of basalt of cement fineness. Water-cement ratio 0.45 by weight, for all ages and admixture contents.

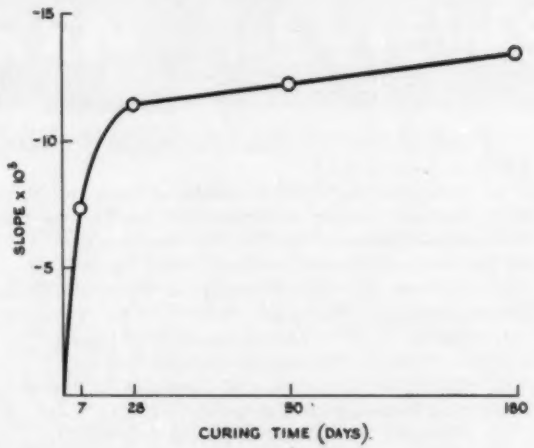

Fig. 10—Slope of graphs of drying shrinkage versus blend composition, for pastes cured under water for 7 to 180 days. Water-cement ratio 0.45 by weight

$$\log \Delta S_{0-R} = \log \Delta x_{0-R} + a' R + b'$$

where

$\Delta S'_{0-R}$ = shrinkage change when admixture content of portland cement is raised from 0 to R percent (by volume)
Δx = accompanying change in the water-cement ratio by volume
R = proportion of portland cement replaced by powdered admixture, on a volume basis
a and b = constants

There is a close analogy between this equation and a previously derived expression relating the accompanying strength changes to changes in the replacement level and water-cement ratio.

3. On the other hand, if the portland cement is partially replaced by powdered basalt at a constant ratio of water to portland cement, the ultimate shrinkage decreases linearly with the sum of the volume percentages of water and powdered basalt in the paste. Further, if the size of the mineral particles replacing portland cement is increased to well beyond that of cement clinker grains, such replacement gives rise to a more rapid and nonlinear decrease in ultimate drying shrinkage, the restraining influence of the aggregate increasing linearly with the mean diameter of the particles.

4. After a 28-day period of water curing, Pozzolan V17 ground to cement fineness altered the drying shrinkage of the portland cement paste in much the same way as basalt powder of similar fineness. Finer grinding of the pozzolan caused a major change in behavior, but only at high replacement levels. A complete change in behavior at all replacement levels was observed when the pozzolan was ground to a very high surface area (85,000 sq cm per g).

5. It is shown that when the effect of aggregate on drying shrinkage is considered in terms of the elastic constants of the embedded particle and the surrounding medium, some distinction should be made between the restraint imposed by rock particles small enough to produce a blended cement by fitting between individual clinker grains, and that imposed by the same

volume of rock in the form of grains or pebbles which are large enough to displace whole zones of clinker grains.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Sept. 1, 1959, for publication in the Part 2, December 1959 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Bridge deck is formed on natural ground—excavation follows

Contractors and Engineers, V. 56, No. 2, Feb. 1959, pp. 80-83

Describes the construction of a twin box-girder bridge using the natural ground as support. The form work and reinforcing steel were placed on a thin concrete slab, which in turn rested on the ground. This reduced the number of cranes and eliminated the need for expensive shoring and falsework. Other construction features are included.

Time element dictates method of constructing thin-shell arch bridge

Contractors and Engineers, V. 56, No. 4, Apr. 1959, pp. 80-87

Describes the construction techniques employed during work on a thin-shell arch, prestressed girder bridge. The five arches, which comprise the central span, were supported by precision placed wood falsework with each arch supported by 170 oak piles. Each span was 172½ ft and had a variable arch thickness of from 20 in. at the base to 14 in. at the crown.

Construction

Explosive buckling of floor corrected in Panama lock overhaul

FRANK H. LERCHEN, *Civil Engineering*, V. 29, No. 3, Mar. 1959, pp. 44-47

The Pedro Miguel Locks of the Panama Canal have been under repair due to deterioration. Separation of the floor slab from the underlying concrete (which contains the filling culverts) has caused considerable damage. The undowelled horizontal construction joint between the lock floor and the foundation was subjected to high, unanticipated water pressure causing the floor to buckle upward in an explosive manner. The culverts, which had developed shear cracks, allowed this water to circulate through these cracks producing numerous voids along the outside of the culverts and under the floor. Article describes the repairs and the cause of the deterioration.

Subaqueous tunnel as construction project (Unterwassertunnel als Bauaufgabe)

BRUNO HAMPE, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 28, Oct. 1958, pp. 1355-1362
Reviewed by ABON L. MIRSKY

Survey of methods of construction of subaqueous tunnels of various types. Illustrated with sketches and photographs of recently constructed tunnels in various countries.

Construction Techniques

Strength of brickwork laid under freezing conditions (in Russian)

W. N. SIZOW, *Novaja Tekhnika i Peredowoi Opyt w Stroitel'stwe* (Moscow), No. 1, 1958, pp. 13-17
Reviewed by N. G. ZOLDNERS

Method of bricklaying at freezing temperatures which has been widely used in Siberia and other arctic areas is discussed.

When brick is laid in freezing weather, the mortar freezes before it sets. The subsequent hardening of the mortar can proceed only during thawing periods. Softening of frozen mortar at thawing may cause movements in the structure.

The height of each stage of the brickwork must be established for each building to in-

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sure the stability of the structure and to avoid shifting of the brickwork during softening periods.

Stability of the brickwork can be improved by placing wire mesh in each of the mortar layers.

Beams and columns must be tied together and centered carefully to avoid eccentricity in load conditions.

Temporary shoring must be erected until mortar has set. The possibility of strengthening the brickwork (if required) during construction work should be considered.

Waterproofing buildings below grade

GRAYSON W. GILL, *Civil Engineering*, V. 29, No. 1, Jan. 1959, pp. 35-37

Multistory basements increase the need for waterproofing against high heads of ground water. General classifications of waterproofing discussed are integral, membrane, and hydrolithic.

Integral waterproofing is done by introducing an admixture into the concrete before it is placed. Its weakness lies in shrinkage cracks.

Membrane waterproofing, consisting of a fabric and a bitumen base applied in layers is theoretically sound but practical usage shows an eventual disintegration of fabric and deterioration of base.

Hydrolithic, or "iron method," waterproofing is applied to walls with good results, and to other surfaces accessible throughout the life of the building. Shrinkage cracking should take place before application.

Joints and cracks in concrete

PETER L. CRITCHELL, Contractors Record, Ltd., London, 1958, 232 pp., 40s

The book is essentially a practical guide to the use of joints in the building of sound structures, although theoretical considerations have not been overlooked. It provides a survey of jointing materials and methods of construction, describes correct and economical jointing techniques, and pays special attention to the diagnosis and treatment of faults. With the aid of more than 50 diagrams and photographs this volume covers all types of concrete structures in which jointing problems are likely to arise, including buildings, water-retaining structures, pavement, bridges, masonry construction, and concrete pipes.

Dams

Modern tendencies in arch dam design

SERGE LELIAVSKY, *The Engineer* (London), V. 204, No. 5316-5318, 1957; Dec. 13, pp. 853-857; Dec. 20, pp. 888-892; Dec. 27, pp. 930-932

Reviewed by ARON L. MIRSKY

A thorough review and critique, aftermath of the 1956 symposium on arch dams of the American Society of Civil Engineers. Current thinking on methods of design and analysis (and use of models) is outlined and discussed (interestingly, the "trial load" method does not fare too well), and well illustrated by plans and details of numerous recent and current dams. Various methods of classification of dams are investigated. Third part of paper is primarily concerned with hydraulic design and its influence on dam type.

Hiracud—India's newest large dam (Hirakud—Indiens neueste Grosstalsperre).

FRITZ HARTUNG, *Der Bauingenieur* (Berlin), V. 33, No. 3, Mar. 1958, pp. 84-97

Reviewed by ARON L. MIRSKY

Dam, recently officially put into service, is 16,000 ft long, built in five sections of earthfill, masonry, and concrete. Current power capacity is 129,000 kilowatts with provision for future addition of 112,500 kilowatts more; a down-stream subsidiary dam will have an ultimate capacity of 96,000 kilowatts; entire project is also of importance irrigation-wise. Article covers all phases of project (for brief English description of project, see *Engineering News-Record*, Mar. 14, 1957, p. 78).

Design

Why measurements on structures? (Warum Messungen an Bauwerken?)

GOSWIN MITTELMANN, *Der Bauingenieur* (Berlin), V. 33; No. 5, May, 1958, pp. 196-199; No. 6, June, 1958, pp. 236-239

Reviewed by ARON L. MIRSKY

After answering the question *why* in convincing fashion (the primary reason being, of course, the proof of the analytical pudding), author presents a fascinating array of field investigations actually completed. They include stresses in prestressing tendons, boundary stresses in concrete shells, interior stresses in concrete pavements, stresses in steel dolphins, vibrations caused by pile-driving operations adjacent to structures,

and pressure variations in the course of underwater demolition using explosives.

Flat plate structure designed by ultimate strength theory

A. Whitting, *Constructional Review* (Sydney), V. 32, No. 1, Jan. 1959, pp. 19-21

Studies the application of ultimate strength theory to the design and construction of a seven story flat slab apartment building. The ultimate shear equation is given followed by an example.

Concealed column heads for mushroom system floors (in German)

L. G. H. Brekelmans, *Ingenieur* (The Hague), V. 10, No. 3, Jan. 1958, pp. 1-14

Applied Mechanics Reviews Sept. 1958

In cases where column heads were undesirable, attempts were made to retain their important constructional advantages by means of steel sections in the floor slab, constituting a concealed head, considering the n times greater modulus of elasticity of steel compared with concrete. Calculations and experiments prove that equivalence could be attained only by a solid steel slab almost as thick as the floor slab. Shearing stresses are certainly absorbed by concealed heads, but braces are more efficient and cheaper for this purpose.

It is recommended that a floor with concealed heads should be considered as a floor without heads according to regulations for mushroom systems in ACI 318-47. In this case a greater thickness of the floor and more steel is to be accepted.

Analysis of elastic frames under dynamic loading using the principle of conservation of energy (in Rumanian)

Mihail Ifrim, *Academia Republicii Populare Romine*, V. 8, No. 2, 1957, pp. 393-406

Reviewed by J. J. Polivka

An analysis of multiple-story frames under dynamic loading using Rayleigh's theory for determining frequencies of free oscillation. Two simple expressions are derived; (1) by considering that the dynamic deformation is in certain relation with the static deformation when framework is exposed to lateral loads; (2) by introducing certain coefficients obtained by assuming that the deflections of the fundamental modes of vibrations vary linearly.

Numerical example of a three-story building with two bays compares the results obtained by an accurate method of analysis with those calculated by these simplified formulas. It is found that in both cases the accuracy is satisfactory, showing a difference of only 1.88 and 0.94 percent.

Theory of highly economical reinforced beams (in Hungarian)

J. Pelikan, *Acta Technica, Academiae Scientiarum Hungaricae* (Budapest), V. 17, 1957, pp. 39-56

Applied Mechanics Reviews Mar. 1959 (J. Barta)

This paper and two previous papers of the author deal with the limit design of continuous reinforced concrete beams. The spans, the load, and the concrete cross section are given. If z is the depth, the problem consists of finding the tensile reinforcement. To solve this problem, author introduces the postulate

$$\int (|M|/z)ds = \min \ldots\ldots (1)$$

Because $|M|/z$ is practically proportional to the tensile reinforcement, postulate leads to a highly economical reinforcement.

In a more recent paper, author treats the same problem and attempts to show that the classical minimizing principle of Castigliano also results in requirement (1). An essential point of this reasoning is the assumption that the flexural rigidity may be computed from the compressed part of the concrete area and from the reinforcement area.

On shells curved in two directions

P. Csonka, Second Symposium on Concrete Shell Roof Construction (Oslo), *Paper* 5, Session II, July 1957, 7 pp.

Applied Mechanics Reviews Jan. 1959 (W. Flugge)

A shell with the middle surface $z = f(x, y)$ carries a smoothly distributed vertical load. The membrane forces can be derived from an Airy stress function satisfying a certain second-order differential equation. The author specifies $f(x, y)$ incompletely, leaving a free parameter A. He assumes the stress function to be a simple polynominal containing two more parameters B and C. Then A, B, and C are found from the condition that the differential equation must be satis-

fied at three well-chosen points, and the result is slightly modified by requiring that the average of the load calculated from the differential equation must equal the average of the actual load. Several simple examples illustrate the method and show that in sufficiently simple cases acceptable solutions can be obtained.

Thoughts on suspended-roof structures (Betrachtungen über Hängedachkonstruktionen)

H. K. Bandel, *Der Bauingenieur* (Berlin), V. 33, No. 6, June 1958, pp. 221-225

Reviewed by Aron L. Mirsky

Comments on some problems peculiar to suspended-roof construction: how to handle the horizontal force components, methods of reducing net spans, roofing, and methods of ensuring stability under unsymmetrical loading.

Electrical analyzer for rigid frameworks

J. W. Bray, *The Structural Engineer* (London), V. 35, No. 8, Aug. 1957, pp. 297-311

Reviewed by C. P. Siess

Describes circuitry and gives examples for an electrical analog for computing moments in frameworks. Device involves use only of resistors and meters. Accuracy of ± 2 percent is claimed with suitable accuracy of electrical components. Factors which can be simulated include wind load, settlement of supports, Vierendeel truss, semirigid joints, effect of axial load, members of nonuniform section, plastic hinges, and elasto-plastic conditions. Can also simulate rectangular frames in three dimensions, continuous arches, and trusses with sloping members.

Tall towers: The design of anchor bolts to resist overturning

Gordon Boyd, *Proceedings*, Institution of Civil Engineers (London), V. 10 (Session 1957-58), June 1958, pp. 193-200

Reviewed by Aron L. Mirsky

An abstract of an unpublished Institution paper giving formulas and graphs for the design of anchor bolts for unstayed slender towers with annular base rings. The effect of the difference in elastic moduli of steel and concrete is taken into consideration. Use of formulas and charts is illustrated by several examples.

Materials

From coal mine waste to light aggregate

Leo Walter, *Rock Products*, V. 62, No. 2, Feb. 1959, pp. 92-95

Describes a sintering process in England where waste shale, by-product of the coal industry, is transformed into light aggregate. The aggregate has consistant uniformity and is available in three grades—coarse, ¾-½ in.; medium, ½-3/16 in.; and fine 3/16 in. to dust.

ASTM standards on mineral aggregates and concrete (with selected highway materials)

American Society for Testing Materials, Philadelphia, 1958, 384 pp., $4.75

The book contains 101 standard specifications, methods of test, recommended practices, and definitions of terms for: mineral aggregates, concrete, concrete curing materials, expansion joint fillers, and miscellaneous related specifications. References are given for specifications on concrete reinforcing steel, paving block and brick, and bituminous and nonbituminous road materials. Of the included standards 46 are new or revised since the 1956 edition, which is superseded.

This publication will be useful to producers and consumers of mineral aggregates, concrete, and highway materials, to highway engineers, specification writers, testing and inspection personnel, and research and engineering institutions.

Heat treatment of high-temperature cements

Technical News Bulletin, National Bureau of Standards, Jan. 1959, pp. 4-5

Reviewed by Robert G. Mathey

Four commercial aluminous cements were investigated. These cements provided material having a fairly wide range of silica and iron oxide content. Test specimens were prepared with a water-cement ratio of 0.25 and heat treated at temperatures up to 1100 C. Laboratory studies showed that after heat treatment many changes in the mechanical properties of aluminous cements are directly related to corresponding changes in compound composition. The grain size, and crystallic and sintering characteristics

of each constituent compound and the solid state reactions between these compounds influenced the mechanical properties in varying degrees.

Regrinding of cement with sand (in Russian)

B. G. Skramtaev, P. F. Shubenkin, A. A. Budilov, *Novaja Tekhnika i Peredovoi Opyt w Stroitel'stve* (Moscow), No. 7, 1958, pp. 19-21
Reviewed by N. G. Zoldners

Portland cement reground for 15 min in a vibro-mill with fine sand, replacing 20, 35, and 50 percent of cement resulted in a finer ground material, specific surface of which is more than 50 percent greater than that of the original cement.

Sand-cement mortar with 20 percent sand replacement showed 20 percent higher 28-day strength and with 35 percent replacement a strength equal to the mortar strength of the original cement. Over 20 percent strength increase was obtained from 28 to 90 days.

After initial 6-hr steam curing the 7-day compressive strength was 87 percent of the 28-day strength as compared with 65 percent for the original cement mortar tests.

Tests conducted on concrete specimens made with sand-cement substantiated the results of mortar tests.

Precast Concrete

Precast concrete in a school building—slender prestressed columns 43 ft high

Concrete and Constructional Engineering (London), V. 54, No. 1, Jan. 1959, pp. 3-9

Reports the use of precast, prestressed columns and beams in the construction of a school in England.

Columns 33 and 43 ft long were prestressed with ¾ and 1 in. diameter center bars, respectively, to prevent cracking in erection. Edge beams, cranked beams, 3 ft deep x 20 ft long I-beams, and the connections of these units are described and illustrated.

Precast folded plate roof for a paint plant

Ignacio Martin and Narciso Padilla, *Civil Engineering*, V. 29, No. 3, Mar, 1959. pp. 56-59

Describes the construction of twelve buildings in Cuba using precast folded plate slabs for the main manufacturing building. The slabs were precast on their companion pieces at the work site and were designed for a free span of 75 ft 7 in. with a cantilever of 7 ft 10 in. on both sides. Two considerations were low-bearing soil and reduction of fire hazards.

Sculptured concrete—an innovation in sand-cast tilt up walls

Modern Concrete, V. 22, No. 11, Mar. 1959, pp. 60-61

Presents information on a recently sculptured concrete curtain wall (24 ft x 11 ft x 6 in.) that was cast in sand as one unit and used for an exterior wall of a restaurant. The process of decorative sculpturing and determining mix proportions is described.

Some effects of prefabrication on postwar buildings

Clifford E. Saunders, *The Structural Engineer* (London), V. 35, No. 8, Aug. 1957, pp. 277-296
Reviewed by C. P. Siess

Comprehensive and profusely illustrated description of numerous types of prefabricated construction for residences and buildings. Includes all concrete as well as steel construction with concrete wall or floor elements. Confined to British practice except discussion of curtain walls which is concerned chiefly with buildings in the United States.

Prestressed Concrete

Prestressing method MR St 150 (Das Spannverfahren MR St 150)

P. Buck, *Bauplanung-Bautechnik* (Berlin), V. 12, No. 5, May 1958, pp. 212-214
Reviewed by J. F. Leppmann

A steel saving method of anchoring post-tensioned bundles of up to 12 parallel 0.196 in. diameter wires. At the anchor end the wires are broomed out and cast into the concrete member to be prestressed. At the tensioning end a 3½ in. diameter hole is cored out of the concrete. In this hole the wires are slightly spread and threaded through and wedged into conical holes which are arranged in a circle in a circular plate. The wires are tensioned from the outside by a pull bolt which is screwed into the cone plate. The prestress is transferred to the concrete through a nut which bears on an external anchor plate. The "socket" is then filled with grout. After the grout has gained a strength of 4200 psi, the pull bolt and anchor

plate are removed for further use. Only the cone ring remains permanently in the concrete.

Prestressed concrete for utilities (Spannbeton in der Energiewirtschaft)

P. MLOSCH, *Bauplanung-Bautechnik* (Berlin), V. 12, No. 5, May 1958, pp. 187-191
Reviewed by J. F. LEPPMANN

Describes prestressed concrete poles used for low and middle voltage transmission lines, concrete footings of wood poles, tubular poles for substations, frames of outdoor transformer stations, and whip-shaped lamp posts. Includes details on shape, strength of materials, shipping weights, fabrication processes, and tests.

Design of prestressed concrete tanks and pipes

HENRY J. COWAN, *The Reinforced Concrete Review* (London), V. 3, No. 8, 1955, pp. 571-585
Reviewed by WILLIAM R. LORMAN

This paper is concerned with derivation of formulas and their application to the design of prestressed concrete containers. Design problems are stated and solutions presented in the form of six examples. The following types of container design are treated: (a) circular tanks and pipes prestressed with continuous wire spiral, (b) effect of shotcrete coating on stress distribution in a tank, (c) tanks prestressed with circular hoops or cables, (d) pretensioned pipes, and (e) design of longitudinal cables for circumferentially wound tanks and for prestressed pipes. Examples show how to account for inelastic strains caused by concrete creep and shrinkage.

Theory of prestressed hollow beams, plates, and shells (in Russian)

V. Z. VLASOV, *Izvestiya Akademii Nauk SSR* (Moscow), No. 5, May 1956, pp. 70-84
APPLIED MECHANICS REVIEWS
Feb. 1959

Starting with the general case, author concentrates on cases representing reinforced concrete problems.

In the case of a beam under transverse bending and torsion, proof is given that, of the three differential equations of equilibrium, only the torsional one is affected by prestressing. The differential equation remains unchanged when GJ (torsional stiffness) is replaced by $G\bar{J}$ (reduced torsional stiffness). Buckling problems are affected in exactly the same manner.

Initial thermal stresses are similarly dealt with, but the differential equation of torsional equilibrium is different, except in one particular case, as it also includes the first derivative of the angle of twist. Both open and closed thin-walled profiles are considered.

Finally, the case of prestressed plates is treated where the prestressing member is either in the middle or off center.

Properties of Concrete

Aggregate-cement reactions in concrete (Les réactions agrégat-ciment dans le béton)

EDMOUR CHAURET, *L'Ingénieur* (Montreal), V. 44, No. 174, Summer 1958, pp. 12-19
Reviewed by ARON L. MIRSKY

Historical summary, followed by comments on ASTM C 227 and C 342 and the results obtained in the Montreal Materials Testing Laboratory with concretes using various aggregates and cements.

Estimating the compressive strength of mortar test specimens

H. KOTTAS and F. UHL, *Schweizer Archiv für Angewandte Wissenschaft und Technik*, (Solothurn, Switzerland), V. 24, No. 3, Mar. 1958, pp. 92-93
Reviewed by PATRICK E. MURPHY

The compressive strength of mortar test specimens prepared according to appropriate standards can be estimated through a linear relationship using values obtained sonically and consequently without destruction. Neither the age of the test specimen nor its W/C ratio must be known.

Experiments conducted for the production of concrete for radiation shielding at the Institute for Building Science (in Hungarian)

GY. KUBIT, *Magyar Épitöipar*, V. 6, No. 3-4, 1957, pp. 126-138
HUNGARIAN TECHNICAL ABSTRACTS
V. 10, No. 2, 1958

A detailed description of tests conducted on the production of three kinds of concrete for radiation shielding of specified weight by volume and hydrate water content to be used in the first Hungarian nuclear reactor under construction. The specified weights by volume were 3.2-4.2 tons per cu m for heavy

concrete and 3.2 tons per cu m for hydrated concrete, specified hydrate water contents were 60, 60, and 140 l per cu m, respectively, the latter being increased to 220 l per cu m. To the first of the above experimental concretes hematite was added as an aggregate, to the second, hematite and iron scrap were added. Experiments covering the third kind of concrete have not yet been completed, the aggregates will be limonite and iron scrap.

Air voids in concrete and characteristics of aggregates

Bulletin No. 196, Highway Research Board, Nov. 1958, $1

Relationship of physical properties of some Iowa carbonate aggregates to durability of concrete
JOHN LEMISH, F. EUGENE RUSH, and CARL L. HILTROP, pp. 1-16

Reports an investigation of Iowa carbonate rocks prepared as aggregates and investigated by petrographic examination. A petrographic study was made of distressed concrete containing aggregate from a condemned quarry. Conclusions are given concerning some good and some bad aggregate characteristics.

Correlation between concrete durability and air-void characteristics
FULTON K. FEARS, pp. 17-28

Thirty-eight beams from 19 mixes were used to study the correlation between each of the five air-void characteristics and durability. A durability factor was used to express the resistance of each beam to deterioration in a laboratory freeze-thaw test. The five air-void characteristics ranked in the order of their correlation with durability are (a) spacing factor, (b) specific surface, (c) number of voids per inch, (d) hypothetical number of voids per cubic inch, and (e) total air content.

The spacing factor and the specific surface were found to be of almost equal importance in producing durable concrete. Hence, either of these two characteristics may be used as a criterion for determining the air requirements for frost-resistant concrete.

Chemical characteristics of some carbonate aggregate as related to durability of concrete
RAMON E. BISQUE and JOHN LEMISH, pp. 29-45

Includes a summary of results of chemical study of some carbonate rocks, one of which has proved to be definitely unsatisfactory. It is used as a comparative standard based on three phases of the investigation. Among the conclusions were: Those carbonate rocks which contained clay, a high percentage of insoluble residue, and high magnesium content, were characterized by masked reaction rims. Pure limestones do not develop reaction rims and appear to be relatively inert.

Hypothesis on normal adherence between steel and concrete caused by intermediate bond layer (in Rumanian)

M. CERNENKO, *Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 7, July 1958, pp. 369-372

Reviewed by J. J. POLIVKA

Theoretical analysis of physical and chemical processes occurring in the formation of bonding interlayer between steel and concrete, corroborated by tests. Of special interest is the increase of ultimate bond strength with time, e.g., in one case after 3 days it was 136 psi, 7 days 238 psi, 28 days 340 psi, and 6 months 430 psi.

Reaction of portland cement with carbon dioxide

CHARLES M. HUNT, VERNON DANTELER, LEWIS A. TOMES, and RAYMOND L. BLAINE, *Journal of Research*, National Bureau of Standards (U. S. Government Printing Office, Washington 25, D. C.), V. 60, No. 5, May 1958, pp. 441-446

HIGHWAY RESEARCH ABSTRACTS Sept. 1958

The reaction of portland cement with carbon dioxide was found to be greatly influenced by the moisture environment of the cement. Reaction with carbon dioxide and water also affected the subsequent hydration of the cement.

In confirmation of earlier observations cement was found to carbonate slowly or not at all when exposed to pure, dry carbon dioxide. In the presence of increasing concentrations of water vapor, however, the rate of carbonation increased rapidly. Cement was also found to carbonate faster when freshly mixed with water than in an atmosphere of high relative humidity.

The amount of nonevaporable water in a hardened cement after 1 day, 7 days, or 28 days of hydration was found to be smaller as the amount of carbon dioxide in the original cement increased. The reduction in the amount of hydration at any given age was greater than the amount of carbon dioxide

associated with the effect, when both were compared on a molecular basis. Water in the original cement also appeared to curtail subsequent hydration of the cement, but the effect was less than the combined effect of water and carbon dioxide.

High-density concrete for shielding in nuclear reactors

D. Campbell-Allen, *Commonwealth Engineer* (Melbourne), V. 46, No. 5, Dec. 1958, pp. 57-61

Discusses the use of concrete for shielding against neutron and gamma radiations from an atomic reactor. Conventional concrete is used except when space is at a premium, then concrete of higher density is called for.

Density of concrete ranging from 150-350 lb per cu ft is obtained by using natural aggregate of barite and hematite for medium dense concrete, and steel aggregate for the higher densities. Mix proportions, strengths, and densities are discussed along with the disadvantages of using this type of aggregate.

Effect of air drying before test—28-day strength of concrete

W. S. Butcher, *Constructional Review* (Sidney), V. 31, No. 12, Dec. 1958, pp. 31-32

Describes the effect of dry curing on the strength of concrete at 28 days. Strength tests were conducted after removing samples from moist curing and allowing samples to dry in air. Length of time specimen was dry cured ranged from a few hours to almost 28 days. Results of the tests are presented.

Influence of shear on the moment of resistance of reinforced concrete beams

K. Subbiah and R. B. L. Smith, *The Structural Engineer* (London), V. 36, No. 11, Nov. 1958, pp. 377-384

The recommendations of British and other design codes on shear in reinforced concrete beams are examined and suggestions are made concerning the basis of an ultimate load approach. Some recent research and theories are discussed. Experimental results obtained by one of the authors are used, together with other published test results, to develop equations for the moment of resistance of beams with or without web reinforcement. The physical significance of these equations is discussed and further research problems are outlined in relation to design requirements.

Concrete for radiation shielding

The Reinforced Concrete Review (London), V. 3, No. 9, 1955, pp. 694-696

Reviewed by William R. Lorman

This article is more than a brief review. It is an abridgement of E. J. Callan's "Concrete for Radiation Shielding" and B. E. Foster's "Absorption by Concrete of X-Rays and Gamma Rays," both of which appear in the Sept. 1953 issue of the ACI Journal. If any reader finds Callan's or Foster's synopsis insufficiently informative and if that reader cannot spare the time required to study the papers, this reviewer recommends the above article as an effective substitute.

Structural Research

Analytical and experimental study of helicoidal girders

Y. F. Young and A. C. Scordelis, *Proceedings*, ASCE, V. 84, ST 5, Sept. 1958, pp. 1756-1–1756-29

Authors' Summary

A study of the helicoidal girder, fixed at the ends, subtending a horizontal angle of 180 deg, and having a slope of 30 deg is presented. Four girders, each having a different uniform rectangular cross section throughout its length, are investigated. The determination of influence lines for end reactions due to vertical load is illustrated using analytical and experimental methods and final results are presented in tabulated and graphical form. End reactions under uniform load over half and full span are also given. The effect of width-depth ratio of cross section on end reactions and internal forces is discussed.

Recent research on deformed reinforcing bars

K. Hajnal-Konyi, *The Reinforced Concrete Review* (London), V. 3, No. 7, 1955, pp. 393-459; discussion pp. 459-476

Reviewed by William R. Lorman

The entire contents of this particular issue are devoted to this comprehensive and important treatise. The extensive series of tests were conducted: (a) to ascertain the comparative efficiencies of cold-worked deformed (Tentor) bars, plain round mild-steel (MS) bars, and square-twisted (ST) bars; and (b) to compare the behavior of Tentor bars, when employed as tensile and compressive reinforcement of rectangular and T-beams, with the performance of MS bars that nor-

mily are used in England for conventional reinforcement of concrete. The Tentor bars were obtained from Denmark because no such bars were produced in England at the time of the tests. The research is divided into four groups: (1) pull-out tests, (2) embedded-rod tests, (3) beams with tensile reinforcement only, and (4) beams with tensile and compressive reinforcement. All references to the "code" pertain to the British Standard Code of Practice C.P. 114.

In his explanation of the pull-out test data, the author shows the superiority of the Tentor bar; the relative efficiency of this type bar increases with increasing load.

The embedded-rod concrete prisms were such that both ends of the protruding bars were pulled and the surrounding concrete tensioned. The author indicates the desirability of developing a new testing method; an improved testing technique is necessary to attain reliable data with the embedded-rod type of test.

The group of tests dealing with beams that only incorporate tensile reinforcement was subdivided thus: large size bars in T-beams and small size bars in rectangular beams. Exact details of loading are illustrated; to suffice this review, loading is approximately third-point. Deflections of beams incorporating Tentor bars are greater than those of beams reinforced with MS bars. For beams containing Tentor bars as large as 0.71-in. diameter, reducing the concrete strength to two-thirds of the minimum code requirement reveals no adverse effect on the safety aspects involved.

The fourth group furnishes proof that beams incorporating plain bars without hooks "...may fail by bond slip before the yield point of the tensile reinforcement is reached." Though hooks are vital in the case of plain bar reinforced beams, the role played by hooks is not exercised until the first slip occurs and generally this results in the concrete bursting at the beam supports; in the case of deformed bars, the beam ends undergo no damage (but the concrete may burst at maximum bending moment). In brief, hooks in conjunction with deformed bars contribute nothing to the safety features of the beams. The author is of the opinion that all structures subject to bending should be so designed that if loaded to failure the tensile reinforcement undergoes yielding, excessive elongation, or fracture.

Section 5 of the treatise is concerned with the lesser crack formation in beams incorporating Tentor reinforcement. Section 6 contains information leading to the best means for improving impact resistance of beams.

Nine short paragraphs in Section 7 summarize the conclusions resulting from the author's extensive work. Conclusion 10 actually is a list of seven needed types of investigation; in the light of current developments, this reviewer considers items 10(c) and 10(d) noteworthy as they reflect, respectively, the need for more information concerning fatigue and creep of reinforced concrete.

Tests on eccentrically loaded columns with square twisted steel reinforcement

R. H. Evans and K. T. Lawson, *The Structural Engineer* (London), V. 35, No. 8, Sept. 1957, pp. 340-348
Reviewed by C. P. Siess

Tests made on 35 tied columns, 10-in. square, with twisted-steel reinforcement having a proof stress of 63,000-69,000 psi. Test specimens were similar to those tested by Hognestad. Steel percentage varied from 1 to 7.6 percent; eccentricities were 2.5, 7.5, and 12.5 in. (axially-loaded columns were included in a previous test program); and concrete cube strengths varied from 2240 to 7500 psi. Results presented and compared with the Hognestad, Stüssi, and Whitney theories.

Direct measurement of stress in concrete and soil (Unmittelbare Spannungsmessung in Beton und Baugrund)

G. Franz, *Der Bauingenieur* (Berlin), V. 33, No. 5, May 1958, pp. 190-195
Reviewed by Aron L. Mirsky

Two newer types of stress meters are described, one (Brosa) a mercury-type meter with points of similarity to the Carlson meter (ACI Journal, Nov. 1952, pp. 201-215), the other (Glötzl) a "safety-valve" type to which most of the paper is devoted. After discussing the effect of the presence of a measuring element with a different relative stiffness on the stress trajectories in an elastic medium, author summarizes developmental tests of the Glötzl

meter. Reports results obtained by its use on the prestressed concrete deck of the Jungbusch Bridge over the Neckar in Mannheim, a composite type structure, where it was possible to study the effects of solar heat on the stresses. Paper closes with the hope that a similar experimental usage with foundation stresses in an actual structure will be possible at an early date—a hope reviewer fervently seconds.

Shear stresses in reinforced concrete beams (in Hungarian)

E. Dulacska, *Mélyépítéstudományi Szemle*, V. 7, No. 5-6, 1957, pp. 201-203

Hungarian Technical Abstracts V. 10, No. 2, 1958

In Hungary the so-called *n*-free method of computation based on the plastic properties of concrete was introduced as standard in 1952. In the specifications however the elastic theory has been retained for the computation of shear stresses. Shear stresses in the present paper are determined from the stage immediately preceding collapse. The determination of shear stresses by both the plastic and elastic methods is investigated for beams having constant and varying reinforcement and formulas satisfying the condition of equilibrium of internal forces are given. Prismatic beams with constant reinforcement cannot be investigated by the plastic theory.

New indirect tensile test for concrete. Theoretical analysis and preliminary experiments

S. Rosenhaupt, A. C. Van Riel and L. Wijler, *Bulletin*, Research Council of Israel (Jerusalem), V. 6C, No. 1, 1957, pp. 13-27. *Road Abstracts*, V. 25, No. 7, July 1958

Highway Research Abstracts Sept. 1958

The tensile strength of concrete cubes may be determined from dimensions of the failure section when a compressive load is applied along the middle of two opposite faces. The problem has been investigated (a) theoretically, by solving the plane-strain problem of a square acted on by two equal and opposite forces; (b) by photoelastic testing of two identically loaded slabs, one square and one circular, and comparing the isochromatic and isoclinic lines obtained; and (c) by testing concrete cubes and cylinders in indirect tension and unreinforced beams in bending, and comparing the results. It is stated that all cubes tested failed along a diametrical plane containing the load, thus confirming the theoretical calculations assigning the maximum tensile stress to this plane; the tensile strength determined theoretically was in close agreement with that obtained by the other two methods; and the tensile strength of prisms was about 80 percent higher than that of cubes and cylinders, which appears to be caused largely by nonlinear stress distribution.

General

Basic geology for science and engineering

Edward C. Dapples, John Wiley & Sons, Inc., New York, 1959, 610 pp., $9.50

Fundamentals of physical geology are clearly and concisely presented. Author presents the basic tenets in logical and interesting style. A useful book for the engineer or student wishing to gain a general knowledge of the subject.

Protecting concrete against corrosion

H. R. Touchin, *Corrosion Technology* (London), V. 12, No. 4, 1957, pp. 417-419; *Building Science Abstracts*, V. 31, No. 2, Feb. 1958, p. 37

Highway Research Abstracts Oct. 1958

The merits and limitations of bitumen or tar solutions, waxes, oils, and chlorinated rubber paints as protective coatings for concrete exposed to aggressive liquids, either hot or cold, are discussed briefly. The properties and optimum uses of coatings based on epoxy resins are also described.

The use of structural concrete in the gas industry

S. V. Gardner, *The Reinforced Concrete Review* (London) V. 3, No. 9, 1955, pp. 601-683; discussion pp. 683-693

Reviewed by William R. Lorman

This interesting and comprehensive compendium, together with the points comprising the discussion, could well serve as a standard reference. Except for three pages, it comprises the entire technical portion of this particular issue of *The Reinforced Concrete Review*.

The main sections are: (1) introduction and historical survey, (2) special physical

chemical conditions, (3) general structural deterioration, (4) superficial repairs, (5) workmanship, (6) construction methods, (7) designs and details, (8) use of prestressed concrete, (9) river and maritime works, and (10) conclusion. In (3) considerable information is summarized relative to chief defects (observed during the last 35 years) and their contributory causes. A good explanation of reinforcement corrosion control is offered in (4). Concrete piles (conventionally reinforced as well as prestressed) are considered in more than a cursory manner in (8).

Specifications for concrete

D. A. STEWART, *The Reinforced Concrete Review* (London), V. 3, No. 10, 1955, pp. 717-727; discussion pp. 727-734

Reviewed by WILLIAM R. LORMAN

This article presents the fundamentals of preparing a specification along the lines of minimum concrete compressive strength, minimum concrete bulk density, and maximum cement content. This procedure insures uniform quality of concrete and permits the contractor to select proportions for the mix, using the most desirable aggregate available locally. The author shows that a worthy specification must provide also for protection of the steel reinforcement against corrosion. Examples of clauses and phraseology are given in a manner such that they could be adapted to nearly any situation confronting the construction engineer.

Discussion pertained to contractors' problems, test specimens, stripping of formwork types of mixers, relationship of concrete bulk density to concrete durability, compaction, and placement problems.

New heavy-duty bitumen composition has remarkable resistance to corrosion

E. A. DULIGAL, *Paint Industry Magazine*, V. 73, No. 8, Aug. 1958, pp. 10, 12, 14, 41

HIGHWAY RESEARCH ABSTRACTS Oct. 1958

A bitumin composition developed in England is claimed to have all the anticorrosive and weather-resistant qualities generally attributed to bituminous paints plus a greatly improved tolerance to heat and greater flexibility at low temperature.

Use of bitugel, as it is called, on concrete has not yet been fully investigated. Results on use as a horizontal dampproof membrane are encouraging, but for vertical use some difficulty has been experienced in adhesion of a cement screed to the dried bitugel film. For maximum adhesion to concrete it is preferable to apply a priming coat of a thin bitumen solution followed by a heavy coat of bitugel which seals hair cracks and gives a uniform, waterproof surface.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirabillty of publishing a list of those periodicals, bulletins, and proceedings being *scanned regularly* for articles dealing with the field of concrete. If all miscellaneous magazines, bulletins, reports, and books were included (which are reviewed only as they come to the attention of the voluntary reviewers or editors) the following list would cover well over 200 periodicals and/or publishing, educational, and research organizations. The following list includes publishers' addresses for convenience should correspondence with them be desired, since copies of articles or books reviewed in the JOURNAL are *not available* through ACI. In most cases they can be obtained directly from the original publishers.

American Ceramic Society Bulletin—American Ceramic Society, 4055 North High St., Columbus 14, Ohio

ASTM Bulletin—American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

Annales de L'Institut Technique du Bâtiment et des Travaux Publics—L'Institut Technique du Bâtiment et des Travaux Publics, 6 Rue Paul Valéry, Paris 16e, France

Annales des Ponts et Chaussées—Commission des Annales à l'École Nationale des Ponts et Chaussées, 28 Rue des Saints-Pères, Paris 7e, France

Applied Mechanics Reviews—American Society of Mechanical Engineers, 29 West 39th St., New York 18, N. Y.

Architectural Forum—9 Rockefeller Plaza, New York 20, N. Y.

Architectural Record—119 West 40th St., New York 18, N. Y.

Archiwum Inzynierii Ladowej—Kultury i Nauki, Pokój 2319, Warsaw, Poland

Australian Journal of Applied Science—Commonwealth Scientific and Industrial Research Organization, 314 Albert St., E. Melbourne, C 2, Victoria, Australia

Bauingenieur, Der—Heidelberger Platz 3, Berlin, Wilmersdorf, Germany

Bauplanung Bautechnik—Unter den Linden 12, Berlin W8, Germany

Bauzeitung, Die—Hospitalstrasse 12, Stuttgart N, Germany

Béton Armé—22 Rue de L'Arcade, Paris 8e, France

Betongen Idag—Norsk Cementforening, Munkedamsveien 3b, Oslo, Norway

Beton i Zhelezobeton—Moscow, Russia

Betonstein-Zeitung—Kleine Wilhelmstrasse 7, Wiesbaden, Germany

Beton-Teknik—Christians Brygge 28, Copenhagen V, Denmark

Beton- und Stahlbetonbau—Hohenzollerndamm 169, Berlin-Wilmersdorf, Germany

British Constructional Engineer, The—32 Southwark Bridge Road, London, SE 1, England

Building Construction Illustrated—5 South Wabash Ave., Chicago 3, Ill.

BRI Abstracts of Building Science Publications—Building Research Institute, 2101 Constitution Ave., Washington 25, D. C.

Building Research Reports, Danish National Institute of Building Research—Danish National Institute of Building Research, 20 Borgergade, Copenhagen K, Denmark

Building Research Station Digest—Building Research Station, Garston, Watford, Herts, England

Builleten Stroitelnoi Tekhniki—Moscow, Russia

Bulletin, Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques—Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, 197 Boulevard Saint-Germain, Paris 7e, France

Bulletin de la Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions—Reunion des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions, 12 Rue Brancion, Paris 15e, France

Bulletin of the Highway Research Board—Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Bulletin of the Institution of Civil Engineers of Ireland—Institution of Civil Engineers of Ireland, 35 Dawson St., Dublin, Ireland

Bulletin del Instituto Técnico de la Construcción y del Cemento—Instituto Técnico de la Construcción y del Cemento del Patronato Juan de la Cierva de Investigación Técnica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Bulletin of the Iowa Engineering Experiment Station—Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa

Bulletin of the University of Illinois Engineering Experiment Station—University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.

Cahiers du Centre Scientifique et Technique du Batiment—Centre Scientifique et Technique du Batiment, 4 Avenue du Recteur-Poincaré, Paris 16e, France

Cement—Herengracht 507, Amsterdam C, Netherlands

Cement and Lime Manufacture—14 Dartmouth St., London, SW 1, England

Cement och Betong—Kolga-huset, Postgiro 1808, Malmo, Sweden

Cemento, Il—Via Benedetto Marcello 2, Milan 405, Italy

Ceramic Abstracts—(see *Journal of the American Ceramic Society*)

Civil Engineering—American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.

Civil Engineering and Public Works Review—8 Buckingham St., London, WC 2, England

Commonwealth Engineer, The—349 Collins St., Melbourne, C 1, Australia

Concrete—400 W. Madison St., Chicago 6, Ill.

Concrete and Constructional Engineering—14 Dartmouth St., London, SW 1, England

Concrete Construction Magazine—P. O. Box 444, Elmhurst, Ill.

Concrete Products—79 W. Monroe St., Chicago 3, Ill.

Construction—92 Rue Bonaparte, Paris 6e, France

Construction Equipment—205 East 42nd St., New York 17, N. Y.

Constructional Review—14 Spring St., Sydney, N.S.W., Australia

Consulting Engineer—227 Wayne St., St. Joseph, Mich.

Contractors and Engineers—470 Fourth Ave., New York 16, N. Y.

Crushed Stone Journal, The—National Crushed Stone Association, 1415 Elliot Place, NW, Washington 7, D. C.

Deutsche Bauzeitschrift—Eickhofstrasse 14/16, Gütersloh, Germany

Engineer, The—28 Essex St., Strand, London, WC 2, England

Engineering—35 & 36 Bedford St., Strand, London, WC 2, England

Engineering Journal, The—2050 Mansfield St., Montreal 2, Quebec, Canada

Engineering News-Record—330 West 42nd St., New York 36, N. Y.

Genie Civil, Le—5 Rue Jules Lefebvre, Paris 9e, France

Giornale del Genio Civile—Via Nomentana 2, Rome, Italy

Gradbeni Vestnik—Erjavceva 11a, Ljubljana, Yugoslavia

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Discussion of a report by ACI Committee 711:

Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)*

By A. C. GRAFFLIN, ANGEL HERRERA, and COMMITTEE

By A. C. GRAFFLIN†

In permitting added load bearing value to be claimed for topping applied over precast floor slabs or decking, it seems to me that Committee 711 has overlooked or ignored the very heart of the matter—the question of ultimate responsibility for the proper application of this topping. Fig. A, typical of several photographs in my possession, shows the actual conditions under which some toppings are applied over precast units. It was claimed that this topping increased the load bearing capacity of these units up to 50 percent over their capacity without such topping, in spite of the added dead load.

I do not see how anyone could claim that topping applied under such conditions, without the slabs being cleaned beforehand, and without the use of either cement or plastic grouting of the slab surfaces or of some mechanical ties of adequate number and strength, could develop any proper bond with the precast decking which would support any claim of such increased load bearing capacity. I do not believe that Committee 711 would support such a claim, under such conditions.

The point is that it seems to be the general practice of the manufacturers and suppliers of floor systems who must rely on and claim this augmented load bearing capacity from applied topping, to have little or nothing to do with its application, and to assume no responsibility for it, even though they may stipulate that it must be applied in accordance with their specifications.

That, to me, is the very heart of what I believe to be the fundamental error in ACI 711-58. No question is raised or involved here whether topping, properly applied, might provide such increased load bearing capacity. What concerns me is who is to be held responsible for this proper application—the architect, the engineer, the general contractor, or, as I believe should be done, the manufacturer or supplier of any system which claims the added advantages of such topping?

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*ACI Journal, V. 30, No. 1, July 1958 (*Proceedings* V. 55), p. 83. Disc. 55-4 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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Fig. A—Conditions under which topping is sometimes placed over precast floor units (unretouched photograph)

I believe that the only honest and responsible and proper basis on which this ACI standard should approve any claims of increased load bearing capacity from applied topping over any type of precast floor slabs made by any manufacturer is: (1) to stipulate clearly the conditions under which such topping must be applied to insure a monolithic bond with the supporting deck; and (2) to stipulate that the topping must be applied by or under the direct responsibility of the manufacturer of any system claiming such added structural advantages and load bearing capacity from the use of such topping in connection with their slabs.

I do not believe that any manufacturer of any precast floor system should be permitted to claim added load capacity for the system through the use of topping if, at the same time, they are in position to completely disclaim any responsibility in the event of failure, on the ground that they had nothing to do with the proper application of such topping.

By ANGEL HERRERA*

On studying this report, a few items have stirred my interest. For example, Section 301 states that floor or roof units shall be tested to complete failure, which shall not occur at less than two times the sum of design dead and live load nor less than three times the design live load. This seems excessive to me for elements which are cast under much better conditions for control of quality, dimensions, and placing material than would be the case for elements cast in place.

There is a certain lack of agreement between this section and the provisions of the section devoted to ultimate strength design in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," which provides for

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design by an ultimate load which shall be 1.2 times the dead load plus 2.4 times the live load, but never less than 1.8 times the sum of both. Therefore, if all elements are designed in accordance with the provisions of the Code, those in which the dead load predominates will be unable to withstand the test satisfactorily, if we accept that the formulas given in the Code for determining their strength are correct.

In those cases where the live load is predominant, the combined factor of safety will vary between 1.8 and 2.4, depending on the relationship between dead and live loads. To reach a factor of safety of 2, it would be necessary for the live load to be twice as large as the dead load, a condition seldom met in practice. Therefore, if we follow the recommendations of this report, it is necessary to design for a greater strength than is required by the ACI Building Code.

Section 408(a) contains certain provisions which do not seem entirely safe to me. The statement is made that precast joists can be keyed a minimum of ½ in. into a field-placed floor slab and may be erected without falsework or jacking and designed for T-beam action with slab, assigning the whole dead load to the precast joist with only the live load to be carried by the combined section of increased depth.

In the case of a roof, the maximum load will possibly occur when the slab is placed, since the joists will have to support their own weight plus the weight of the forms and slab in addition to the live loads produced by the workers and equipment, such as vibrators, buggies, etc. These extra loads can easily be more than the 10 or 15 lb per sq ft which it is usual to employ as roof loads.

For the above mentioned reasons, I believe that the joists should not be considered as T-beams, but should be checked for both dead and live loads existing at the time of casting and until the time the concrete slab can contribute to carrying the load. They should be checked as T-beams for the dead load which will act after the concrete has set, as well as the design live load, to furnish strength necessary under the most unfavorable conditions.

For the first check, lower factors of safety may be employed than for the second one, since these are temporary loads and failure at this time would not have serious consequences.

These suggestions are also applicable to the provisions of Sections 411 and 413.

COMMITTEE CLOSURE

The question of responsibility for a construction operation often leads to inferior construction, misunderstandings, and litigation. Ideally, all questions of responsibility should be answered by the terms of the contract and specifications under which a structure is built.

Most structures are built by a general contractor working together with a number of subcontractors. Where one subcontractor works on a surface furnished by another, as pointed out by Mr. Grafflin, the question of re-

sponsibility for the condition of the surface arises. It is common practice to require that, before proceeding, the subcontractor accept or reject the surface on which his work is based. His acceptance makes him responsible for the surface condition. His rejection requires that the general contractor take the necessary steps to correct the condition. The specifications should place the responsibility for this work.

In the case of structural concrete topping on precast elements, the surface of the precast floor is usually good as installed, with ordinary cleaning. However, the work of other subcontractors, improperly supervised, often results in damage to the surface. The manufacturer of the precast system has no authority over those subcontractors, and cannot therefore, protect the precast surface. The general contractor has such authority and should use it to see that the work of one subcontractor is properly treated by others.

No standard, such as the one discussed here, can by itself guard against poor construction practices. The proper cleaning of topped surfaces is a requirement of the standard which must be enforced by proper inspection backed up by adequate construction specifications.

It is assumed that the manufacturer of the precast system has proved, by proper certification (Section 302, ACI 711-58), that the topping can be bonded to the precast member by a procedure such as that outlined in Section 403.

Mr. Herrera mentions that there is a certain lack of agreement between the failure load requirements of Section 301, ACI 711-58, and the load factors in Section A604 of ACI 318-56. Since it is intended that precast members may be proportioned by ultimate strength design, there seems to be little reason why the test requirements and the ultimate strength requirements should not agree. This difference has been noted and will be further considered by Committee 711.

Referring to Section 408(a), if a roof is designed for a small live load it certainly is possible that the construction load applied to the joist only might control the joist design. However, it is not considered good practice to design roofs for a live load as low as 10 to 15 psf as stated by Mr. Herrera.

Discussion of a report by ACI Committee 622:

Pressures on Formwork*

By CHARLES MACKLIN, M. R. MONTGOMERY, ROLF SCHJÖDT, WASSIL WELEFF, and COMMITTEE

By CHARLES MACKLIN†

Fig. A shows concrete pressure curves for temperatures of 50 and 70 F as reported by Shunk (1909),[21,22] Meyer (1913),[23] Pulver (1933),[24] and this report by ACI Committee 622, with additional curves by Rodin (1952)[5] and Macklin (1946)[7] considering hand spading and vibration only.

The work of Shunk considered only rate of placement and temperature. In 1933 Pulver introduced the formula $P = 150 + KR$ which was copyrighted by Universal Form Clamp Co. and widely circulated by the Portland Cement Association, reaching the attention of Rodin who constructed the chart giving the percentage correction for temperature. The Pulver temperature K was in terms of an equivalent fluid of 150, 120, 100, and 90 lb per cu ft for temperature steps from 50 to 80 F by increments of 10 F. The Meyer (Shunk) tests had a time factor (20, 25, 35, 50, and 70 min.) for temperatures from 80 to 40 F in increments of 10 F. The similarity of the two curves is apparent from Fig. A.

The formulas shown in Fig. A as straight or slightly curved lines are based on the supposition that termination of the fluid behavior of concrete is dependent on the chemical action of the cement which in turn is dependent on temperature and type of cement. This hypothesis has some apparent errors which are clearly demonstrated by slip-form work, vacuum concrete, and the tests by Roby.[25]

I would like to offer another hypothesis which assumes that the solids in concrete are temporarily suspended in a grout. This grout shows liquid characteristics for a variable time dependent on many factors, some of which have been studied and reported.[26-28] The solids, usually of higher specific gravity, sink in the grout at a uniform rate and have a measurable space between them which is filled with grout, air, and clays. When the lowest layer lands on the bottom it stops, with successive layers coming to rest and building up a precarious balance of solids of various shapes. Spading and internal or external vibration causes the solids to come to a state of less precarious stability and the water and lighter particles work toward the top. During this process chemical action is progressing dependent on the tem-

*ACI Journal, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), p. 173. Disc. 55-10 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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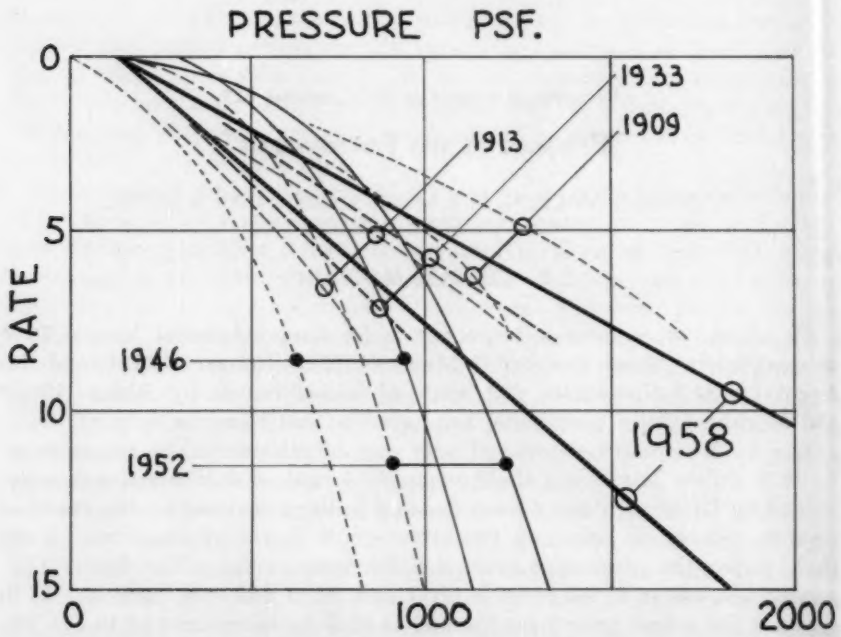

Fig. A—Solid lines indicate concrete placed with mechanical vibration; dotted lines indicate concrete placed with hand spading. Straight line values are all based on the experimental data of Shunk, temperatures of 50 and 70 F. Curved lines are modifications of the Smith formula and do not consider temperature

perature and type of cement used. The time taken for the solids to settle is due largely to the amount of cement and its consistency. Lack of cement causes complete and rapid settlement of solids in water. As cement is added it takes longer to effect contact between solids. This is borne out by the Smith tests.[29] An increase in cement causes an increase in pressure; an increase in water causes a decrease in pressure, which was also determined by Smith. These two facts are compatible with the hypothesis that concrete pressure is terminated by solids making contact. The effect of the chemical action in the cement can lead to delayed settlement. Teller[30] emphasized the small effect of temperature on pressure. Vibration of sufficient energy dislodges the solids and causes temporary fluidity which causes a virtual equivalent fluid pressure. This, however, diminishes rapidly as soon as vibration stops. Revibration has been reported by Tuthill and Davis[31] to be successful after several hours and all vibrated concrete has a greater weight than the hand spaded sample.

The committee report is an excellent step toward solving this important and controversial problem and it has shown clearly the present conception and usages of concrete pressure values.

A complete bibliography would be useful and reports of experiments interesting.

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By M. R. MONTGOMERY*

This report has condensed available information into a useful and practical form for ready reference and should consolidate general thinking on the subject. I do not intend this as a technical discussion in a formal sense since my observations are based on experience and tests made a number of years ago for the specific purpose of checking form safety design on very heavy forms. The test data are no longer available; however, it did have a definite analogy to the data presented in the committee report.

Most form pressure tests are based on the theory that the form is a rigid body and does not deflect under load, which is theoretically correct but never obtained in any but special cases, such as a circular steel column form. Wood to wood and wood to steel bearings compress under the normal form clamps. Deflection between studs and between walers occurs even though in small amounts. Large steel forms designed for heavy construction with trusses and channel walers obviously cannot be economically designed to completely prevent deflection.

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This was borne out in a series of large scale tests conducted in connection with form pressures and form design for large movable steel forms for sloping side walls of a spillway structure. Briefly, the forms were cantilever trusses anchored to the floor slabs and concreted in alternate sections with the "fill-in" forms tied down at the top to insure alignment. Deflection was estimated for the first placement by calculation and later by actual measurement and allowed for in setting the free standing forms.

Pressure cells were inserted at various points on the face of the form and independently mounted vernier micrometers of the plunger dial type placed on the outside of the forms to measure deflection. Pressure cells were also inserted at the same elevations on the ends of the previously cast walls where no deflection was possible.

The cells mounted on the solid concrete followed closely the hydrostatic heads at 150 lb per ft for up to 2 hr of concreting, which was normally at a rate of 4 to 6 ft per hr under heavy vibration. The face form pressures, however, built up as expected for about 30 min or 2 ft of head, whichever came first, on a straight line regardless of deflection. From then on pressures increased in an inverse ratio to the measured deflection, all other things being equal. Pressure usually dropped to nearly zero at the end of 1 hr with deflections at any point approaching ¼ in.; however, there was never any indication of the form actually leaving the face of the concrete.

Considerable thought was given to the relation between deflection and form pressures since it appeared that work energy entered into the picture due to resistance of the concrete to change of shape when deflection, no matter how small, occurred. Varied rates of placement did not effect the pressures directly as would be expected based on time of set but seemed rather to be partially compensated by an increasing angle of friction due to internal pressures from vertical loading. Since the tests were made primarily to investigate the safety of the form and tie down designs and proved to our satisfaction that deflections within the system would compensate for any variations in rate of placement, temperature, etc., under normal operating conditions, no further testing was carried out. Informal discussions with soil engineers on the project did indicate that our results for initial periods were consistent with what might be expected from uncemented granular soil masses under the same conditions.

I believe that the results of the above limited tests at least indicate a consistency between the USBR column tests, which produce a straight line increase; Rodin's and Macklin's experimental curves for maximum pressure, which I assume were on normal wall forms and square columns where some deflection occurs; and Economy Form Co.'s designing around deflection of members.

The writer believes that the recommended formula is excellent for rigid column forms under high rates of placement and for architectural concrete where highly plastic mixes, relatively slow rates of placement, and forms designed to prevent deflection are a normal requirement. It is believed,

however, that for "engineering concrete," the loading is probably a moving triangular load with the maximum pressure at the effective depth of vibration and the shape of the triangle dependent on the angle of friction of the consolidated mass under the load of concrete above and the permissible deflection of forms. Tests and engineering data developed by the large form manufacturers surely will tend to prove or disprove this theory.

By ROLF SCHJÖDT*

ACI Committee 622 has made an interesting study of the variables influencing concrete pressure on formwork, and given an illuminating comparison between the various formulas proposed for calculation of the pressure.

For the pore water pressure, the committee remarks that "this variable may well prove to be a better approach to the more accurate resolution of lateral pressures, but the committee is hesitant to recommend it until more basic research has been done."

In the opinion of this writer it is impossible to arrive at a formula for form pressure which is even approximately correct under all circumstances without taking into account the pore water pressure. When all factors are equal, with the exception of this last, the maximum pressure on the formwork may vary between, for example, 150 and 500 psf. This difference can not be found by calculation if the pore water pressure does not enter into the formula.

In the writer's ASCE paper[1] it was considered necessary to use the value for the depth of the vibration effect in the formulas. This made them more complicated, and moreover, this value is difficult to fix with any certainty where the concrete is vibrated.

Later experiments showed that the above is not necessary, as the depth of the vibration effect appears in the coefficient λ, which gives the relation of lateral pressure to vertical pressure (and also in κ). In the two curves shown in Fig. B and C it will be seen that the vibration was sufficiently strong to transform the concrete to the liquid state after 2 and 2.5 hr for a depth of concrete of 5 and 2.6 ft.

The formula now becomes much simpler. Thus, using the same notation as in the committee report, the formula for maximum pressure cited by the committee becomes:

$$p_{max} = \tfrac{1}{4}\left[\gamma\lambda + \gamma_o\kappa(1-\lambda)\right]\left(1 + \frac{1}{\lambda}\,\frac{\gamma_o\kappa}{\gamma - \gamma_o\kappa}\right)Rt$$

However, for practical use this can still be simplified, if we enter with the values of the symbols. Thus, we can write for vibrated concrete (with $\lambda = 1$)

$$p_{max} = \left(38 + \frac{2400\,\kappa}{150 - 63\,\kappa}\right)Rt$$

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and for nonvibrated concrete ($\lambda = 0.5$)

$$p_{max} = (19 + 7.8\,\kappa)\left(1 + \frac{125\,\kappa}{150 - 63\,\kappa}\right)Rt$$

The value of κ, the ratio of pore water pressure head to height of concrete above the point in question, can be nearly zero for leaky forms and can reach the value of 1.0 as the other extreme. Usually it is safe to take 0.8 as the upper limit. R is the rate of placement and t is the setting time for the concrete. The latter depends on the temperature and the quality of the cement. The writer considers it a weakness in the formula proposed by the committee that only the temperature is considered. As is well known, there is a great difference between the setting times of the various cements and using, for example, calcium chloride or gypsum in the mixture will change this time considerably, and with it the pressure. Even for cements designated as "normal," the setting time can vary greatly.

It is easy to measure pore water pressure, which will also give information about the setting time and about the depth to which the vibration effect reaches. Fig. D shows how these measurements are made and the simple equipment needed.

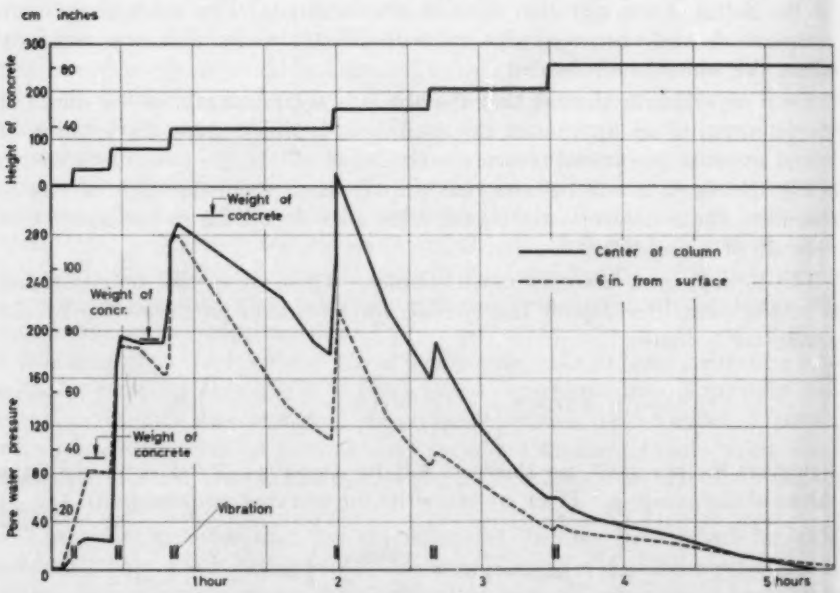


Fig. B—Pore water pressure in 40-in. square column. Note rate of placement

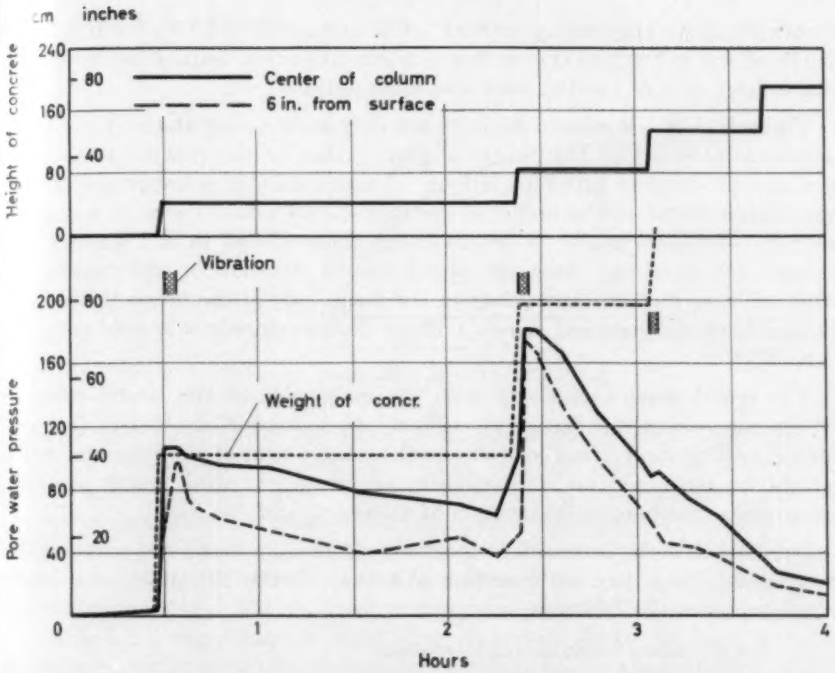
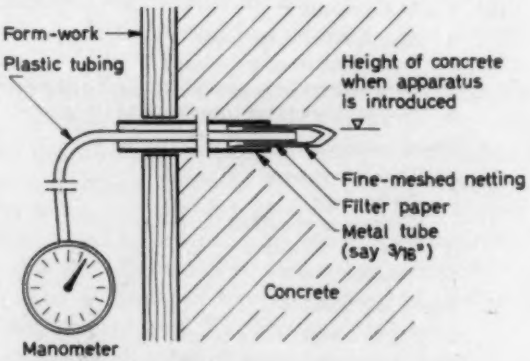


Fig. C—Pore water pressure in 37-in. square column. Concrete pressure is greater than the pore water pressure

Fig. D—Apparatus for measurement of pore water pressure



By WASSIL WELEFF*

Considering the rate of major and minor accidents in recent years occurring during the building of concrete projects, the importance of the work of Committee 622 is more timely and necessary than ever before.

It is of great importance to the public welfare that engineers fill the gap in most of the building codes with provisions for design and construction of

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formwork in an engineering manner. The committee is to be congratulated for its efforts in bringing this matter to public attention, helping in stimulating discussions, and in making some recommendations.

The safety of a structure depends not only on the magnitude of the loads assumed to be acting but also to a great extent on the manner these loads are carried down to safe foundations. Undoubtedly it is important to find the magnitude of *all* the probable, accidental, and actual forces as well as to provide adequate means of transmitting their effects to the ground. If failures are occurring, these are mostly due to oversight or underestimating some of these factors, rather than to the magnitude of the forces themselves. Once a force is recognized, the duty of the design engineer is to take adequate care of it.

The report deals exclusively with the magnitude of the lateral pressure of the concrete on the formwork. There are a number of other factors and forces, next to the vertical load due to the dead weight of the concrete, which should be accounted for in the design of formwork: wind, earth pressure, earthquake, mechanical destruction of members, and the like.

It is felt that the committee report should have outlined the scope of the recommendations they are intending to make. Briefly the provisions should be classified in the following groups:

A. Scope—General information and limitations
   a. Purpose of formwork
   b. Material used: steel, wood, plastic, etc.
   c. Inspection

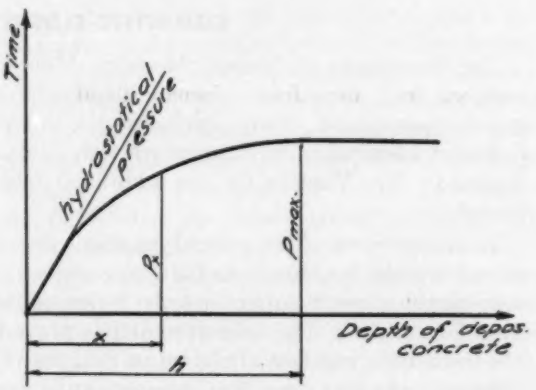
B. Loading conditions
   a. Assumption for the loads
   b. Factors influencing the loads
   c. Load distribution (account for placing of concrete on one side of the structure)
   d. Manner of carrying these loads to the ground

C. Design provisions
   a. Allowable stresses
   b. Connections
   c. Limitations regarding the deflection
   d. Causes of deflection
   e. Stability

D. Safety provisions
   a. For formwork above ground
   b. For formwork below ground
   c. For adjacent structures

Without going into detail it is assumed that certain work listed above is done or intended to be presented by the committee in the future. It may be helpful to engineers, however, to know the extent of the committee work, to allow them to concentrate their discussions on the presented subject only, and not to repeat suggestions which may be already a part of the basic work of the committee.

Fig. E—Effect of time on form pressure



Under the above classification the report covers only a part of paragraphs B(a) and (b), namely: loading conditions, assumptions of the loads, and factors influencing the loads. Before discussing the committee's formula and recommendations, a general discussion of *all* possible, actual, and accidental loading conditions for a broad variety of formwork would be desirable. There are entirely different forces acting upon a formwork for, let us say, concrete pavement, or flat slab construction, or a grain silo. A careful analysis should be made regarding the possibilities of certain accidental loads such as wind loads, water pressure, ice pressure, earthquake, construction live loads, dynamic effect of rolling material, explosions, and destruction of certain members. Consideration should be given those loads and recommendations made regarding the assumptions of their magnitude and, provisions should be made for their safe transmittal to the ground to safeguard the structure against damage or destruction.

The formula recommended for the pressure on the formwork is truly simple but it tends to confuse the real problems involved. The variables are reduced to a minimum compared with the recommendations of Rodin, or Hoffman, but there are other limitations imposed, such as the arbitrary rate of placing concrete of 7 ft per hr and the maximum values of 3000 psf for columns and 2000 psf for walls. The simplifications on one hand are curtailing the clarity of the problem and the introduction of some new coefficients (C_1 and C_2) are actually complicating it. The tendency of having simple and practical formulas is good but should not be permitted if confusion is invited.

Another item which should also be considered in the proposed formula is the time element. It is known that the concrete mix hardens when time elapses. Freshly mixed and deposited concrete behaves more like a liquid mass (see Fig. E). The hardening process starts pretty early and is different for different concrete mixes. Thereafter the concrete mass behaves more like a rigid body and the formwork pressure is maximum at a certain time. It is therefore not necessary to consider the whole formwork pressure for the entire depth of high walls or columns.

COMMITTEE CLOSURE

The discussions of Messrs. Macklin, Montgomery, Schjödt, and Weleff, together with numerous informal discussions and letters of members are greatly appreciated. They emphasize the interest and importance of the assignment undertaken by Committee 622. In addition the committee is indebted to Mr. Macklin for the additional bibliography and history he has furnished.

To answer some of the general questions raised in several of the discussions we will restate the objective of the committee. This is simply: "to report on and suggest a specification, or code, for recommended good practice for concrete formwork." The achievement has proved a trying and difficult task. The committee was first organized in 1955 and Harry Ellsberg was appointed chairman. At that time nine subcommittees were appointed for the purpose of correlating data and studying recommendations for the design, construction, and inspection of formwork. To answer Mr. Weleff's questions they are:

Subcommittee No. 1—Pressures on Formwork
Subcommittee No. 2—Construction Practices
Subcommittee No. 3—Preparation of Comprehensive Recommended Practice
Subcommittee No. 4—Form Materials
Subcommittee No. 5—Bridges
Subcommittee No. 6—Mass Concrete, Prepacked Concrete
Subcommittee No. 7—Shell Structures and Precast Concrete
Subcommittee No. 8—Architectural Concrete
Subcommittee No. 9—Underground Structures
Subcommittee No. 10—Concrete Form Failures

The above committees regularly report on their respective programs; when recommendations are formulated they are published as was done in the report "Pressures on Formwork." The majority of Mr. Weleff's comments pertain to the work of Subcommittee No. 3, and we welcome them and similar suggestions from anyone interested in our problem. Published reports of Subcommittees No. 2 and 10 are also available from ACI headquarters,[32,33] and information on the current progress of the others may be had from Mr. Ellsberg.

We believe an error in the published equations on p. 189 has contributed toward Mr. Weleff's conclusion that they are confusing. The formulas in the middle of the page should read:

$$p = C_1\left(1 + \frac{C_2R}{T}\right)$$

and

$$p = 150\left(1 + \frac{60R}{T}\right)$$

The following questions can be answered by repeating or clarifying basic assumptions made in the report:

(a) It has been called to our attention that the committee's report implies that deep vibration, revibration, and external vibration are inadvertant malpractices which should be avoided. This was not intended. Currently developing reports of ACI Committees 605, 609, and 614 make reference to the benefits of revibration, in some cases including vibration at depth. Normal vibration, generally considered to be effectively complete when considerable amounts of air cease bubbling to the surface and mortar just begins to flush to the top,[31] has been included in the recommended formulas. We believe forms designed with these formulas will be adequate for revibration as long as the concrete remains in a fairly plastic state, say up to 2 hr after mixing, and the revibration does not extend to depths greater than about 4 ft. The committee does not have data on form pressures where revibration at depths greater than 4 ft has been practiced. One reason for an increase in pressure is discussed in a later paragraph. External vibration, far from being a malpractice, is the accepted norm in many cases, such as in the manufacture of concrete pipe. However, this formwork is designed accordingly, and the proposed formulas do not apply.

(b) We agree with Messrs. Schjödt and Macklin that the type of cement and the use of admixtures will vary the setting time and therefore the lateral pressures. However, on p. 177 under "Type of cement" we stated that Type I cement was used by most investigators and, therefore, would form the basis for the conclusions and formulas recommended. We do not have sufficient data, at present, to include the effects of this variable.

(c) The formulas proposed in the report represent the *maximum* lateral pressure at any elevation in the form under the prescribed conditions of temperature, rate of placement, type of cement, vibration, weight of concrete, and slump. This, we believe, is the pressure to be used in designing formwork. Mr. Montgomery's belief that the loading is a moving triangular load, Mr. Weleff's recommendation to include a time element, and Mr. Schjödt's equation which gives the pressure at any point, offer solutions of general interest and further the understanding of lateral pressures. We believe variation of pressure at a given point with time to be beyond the scope of the committee's needs, and again recommend Mr. Schjödt's paper[1] as the most informative on this subject.

(d) Although it was not clearly stated in the report, the formulas and discussion pertain to structural concrete. As noted by Mr. Montgomery, engineering concrete (we have termed it mass concrete) has properties and requirements with regard to formwork that deserve special consideration. His very helpful discussion has been called to the attention of Subcommittee No. 6.

Other questions can best be answered by an additional discussion of the internal properties of the mix and the effect of the mix on the form.

Mr. Macklin's hypothesis concerning the action of solids suspended in grout is an excellent one in view of current knowledge and agrees well with that discussed by Messrs. Tuthill and Davis. We would like to add that

perhaps the reason that the solids are suspended in grout for an appreciable time is the inability of the free water in the grout to readily escape from the voids between the aggregate and cement particles. With time, some water is absorbed by the cement particles and some seeps through the matrix, thus reducing the pore pressure so that the entire superimposed weight of the mix is eventually carried by contact between the solid particles. Further vibration results in greater densification of the solid matrix and returns the pore pressure to the maximum value because of the decrease in volume of voids. Finally, the solids are in the most stable configuration possible for the construction method being used, pore pressure decreases as the cement is hydrated, and insufficient water is present to fill the voids. Such a hypothesis explains the effect of pore pressure as measured by Mr. Schjödt and the increasing angle of friction discussed by Mr. Montgomery. If we assume that the forms are reasonably watertight, as stated in the report, Mr. Schjödt's factor κ is equal to unity. If the forms are not reasonably watertight, the factor κ is less than unity and can only be evaluated after additional research has been done, as suggested by the committee.

Since the form design must be based upon the greatest pressure at each point (actually the envelope of maximum pressure along the form) the variation of pore pressure with time at a particular point is somewhat academic for the problem at hand and will not be discussed further.

Although a knowledge of what is happening in the concrete mix is highly important to explain the various test results, the action of the mix against the form is the problem with which we are dealing. Consolidation of the concrete is not the result of vibration alone or of the superimposed weight of the mix alone but is due to a combination of the two. In addition, internal mechanical vibration not only results in a more stable arrangement of the aggregates but also causes a wedging action tending to force the forms apart. Therefore, concrete confined in a narrow space by forms will exhibit different properties than mass concrete which might be thought of as being restrained along a semi-infinite boundary. We have recommended that the proposed formula be used only for column and wall forms with controlled rates of placement and plastic mixes, with which Mr. Montgomery agrees, but have left the problems of mass concrete for another subcommittee. As the mix rises in the forms, the total pressure spreads the forms laterally until the entire form and form ties adjust to carry this load. The deflection of the form, a function of the pressure from the mix and the elastic properties of the form, is more pronounced in cantilever forms than in column or wall forms, as discussed by Mr. Montgomery. Finally, as pore pressure decreases, the total force on the form decreases, but the form is held in a deflected position by the concrete matrix. The fact that force is required to loosen form tie nuts or clamps indicates that the pressure does not go to zero.

Assuming that vibration is complete along the form and that the form deflects to resist the load, the compression which occurs in walers, studs, and shims is not important if the major portion of the deflection occurs as the

load comes on the form. Erratic tie rod tensions which complicate an analysis of the results could be caused by overtightening of some rods, thus loosening others; weak or improper splices of studs, walers, or lateral bracing against the forms; and localized vibration of the mix.

In view of the above, we feel that Mr. Macklin's statement: "The formulas shown in Fig. A as straight or slightly curved lines are based on the supposition that termination of the fluid behavior of concrete is dependent on the chemical action of the cement..." is incorrect. The formulas shown are based on field or laboratory measurements of actual pressure on the forms and do include the effect of all variables, but the results have been plotted in terms of rate of placement, temperature, and pressure only.

We also feel that Mr. Macklin's statement that an increase in cement causes an increase in pressure and that an increase in water causes a decrease in pressure is partially correct and needs further clarification. If a mixture of aggregate and water were to be placed in the forms as might occur with prepacked concrete, point contact between the coarse particles would be immediate, and the lateral pressure against the forms would consist of the hydrostatic pressure from the water and the active pressure from the buoyant aggregate. This pressure would be a maximum in the lower portion of the form, would be a function of the total height of the placement, and would be independent of the time involved.

On the other hand, if the same procedure is repeated with cement particles added to the mixture, water will become trapped in the voids as the concentration of cement increases so that the aggregate is temporarily suspended in the grout, thus increasing the pressure in the grout which in turn increases the pressure on the form. However, the point of maximum pressure is no longer near the bottom of the forms but actually moves up the form depending on the rate of placement and temperature. Thus the location and magnitude of maximum pressure are in no way related in the two procedures. Moreover, a point would be reached when the addition of cement would have little effect, especially since its dispersing properties are counteracted by setting qualities.

We do not agree with Mr. Montgomery that "most tests are based on the theory that the form is a rigid body and does not deflect under load." Most tests have been made with the use of various types of pressure cells to measure the pressure acting at selected points, and such pressures must include the effect of deflection of the forms. Mr. Montgomery's statement, "Since the tests were made primarily to investigate the safety of the form and tie down designs and proved to our satisfaction that deflections within the system would compensate for any variations in rate of placement, temperature, etc., under normal operating conditions, no further testing was carried out," is informative but is not applicable to propped cantilevers and panel type forms or where retarding agents are used.

DATA NEEDED IN THE FUTURE

Since we are actually dealing with form design, we should be concerned with the load on the form. The committee felt that a scheme using measurements from pressure cells was not as suitable as one gaging the actual tensions in the tie rods. With the total load known, a suitable distribution of lateral pressure, such as uniform pressure on a horizontal line, could be assumed, and the form could be designed. Because of possible variation in initial tie rod tensions, elastic properties of the form, location of splices, and other variables, we cannot expect to obtain results from a few tests. However, as mentioned in the report, a large number of future tests would serve to establish an envelope of pressure or perhaps a weighted value of pressure if the latter is more desirable.

The Research Division of the Hydro-Electric Power Commission of Ontario has developed a measuring device to be mounted directly on the tie rod. This device is being used in Canada, and it is easy to install and to measure the load. It consists of a heavy U-shaped member which is mounted on the tie rod and deflects as the lateral pressure is applied. A portable dial gage is used for measuring the deflection of the open ends of the U-shaped members.

The committee hopes that this, or a comparable device, can be made available to the construction industry, perhaps by loan from ACI or other interested groups. In this way a limited number of tie rod tensions could be obtained on any form and the data could be collected and analyzed by one agency.

By means of this device the questions pertaining to external vibration, deep vibration, type of cement, and pore pressure could be reconciled. Then the proposed formula, which is presently based on current practice and tests to date, could be expanded or revised as necessary.

In closing, the committee recognizes the shortcomings of the proposed formulas as reiterated in this discussion. However, it did not seem practical to delay the adoption of a uniform standard which we felt would give satisfactory results. Moreover, it serves as a basis for present study and future modification. We feel that the following statement by Shunk (*Engineering News*, Sept. 9, 1909)[21] pinpoints our problem:

> "I am far from claiming absolute accuracy for the results obtained, but I think they are sufficiently exact for the purpose, which is not to establish a theory of molecular physics, but only to find a practical rule for determining strength of forms or rate of filling."

REFERENCES

32. Subcommittee 2, ACI Committee 622, "Form Construction Practices," ACI JOURNAL, V. 28, No. 12, June 1957 (*Proceedings* V. 53), pp. 1105-1118.

33. Feld, Jacob, "Failures of Concrete Structures," ACI JOURNAL, V. 29, No. 6, Dec. 1957 (*Proceedings* V. 54), pp. 449-470.

Discussion of a paper by A. C. Scordelis, T. Y. Lin, and H. R. May:

Shearing Strength of Prestressed Concrete Lift Slabs*

By A. H. BROWNFIELD and AUTHORS

By A. H. BROWNFIELD†

The writer initiated this testing program by a memorandum to C. M. Herd, chief construction engineer of the Division of Architecture, on July 18, 1955. The necessity for such a program became obvious due to a lack of actual test data while reviewing a contractor's proposal to construct a prestressed lift slab at Vacaville, Calif.

My comments will be limited to testing procedures as observed by a structural engineer rather than from the point of view of a research engineer. As a sidelight, one of the basic differences of opinion was the matter of ultimate loads and failures. To the writer, a test structure has failed when it becomes useless or hazardous. The researchers desired to carry all tests to destruction. After observing several tests, we might define the useful limit of a structure as the point where the laboratory starts to remove their valuable testing equipment from below the specimen being tested. The following comments are not intended as criticism.

It is easy to propose a testing program, but from that point on it becomes a problem in cost, number of tests, time, and results to be expected. The module was established to conform to previous tests, so that comparisons could be made. This in turn limited the collar size, and it was made sufficiently rigid to serve several times during the program. The collar also needed to be rigid so as to eliminate its effect on the model, as we were primarily interested in testing the concrete rather than the stresses in the collar. We feel that fabricated collars should be of nearly uniform stiffness around the perimeter of the columns to avoid local overstress in the concrete and avoid cracking.

The scope of the tests was established after several conferences, since we were desirous of specific information in a rather limited useful range. The researchers desired a broader field for establishing valid limits when drawing conclusions. Some specimens were underreinforced and others overreinforced, in terms of what we would design on our contract plans. The method of reinforcement necessarily varied somewhat from field placement due to the small specimen size. The supporting frame was quite rigid and the specimens deformed as usual, so that they approached a condition of a short strip

*ACI JOURNAL, V. 30, No. 4, Oct. 1958 (*Proceedings* V. 55), p. 485. Disc. 55-32 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Supervising Structural Engineer, Division of Architecture, State of California, Sacramento, Calif.

loading on four sides, which appears unrealistic. Later tests indicate a concentration of moment at the columns, and in an actual slab this could be the full width of these specimens, rather than the center portion.

If further tests are made for this type of structure, some thought should be given to larger circular specimens tensioned in a bed to maintain the usual square pattern for the strands. A circular loading collar and loading frame would probably be more realistic than the square frames used, since the square collar frequently approaches too close to the edge of the specimen being tested.

The writer feels that the tests served a useful purpose in spite of their shortcomings, and that the conclusions are realistic and perhaps conservative for short-time loading. This is somewhat confirmed on actual structures where unknown loads are frequently imposed on the slabs during the lifting operations. The tendency for lift slabs to hang up on dowels, which impose local deflections while lifting, has been noticeable on occasions. No tests have been made with large holes adjacent to the collars, and this practice should be discouraged until factual data are available.

AUTHORS' CLOSURE

The authors appreciate Mr. Brownfields' comments on their paper, especially since he took such an active part in the planning of the research program. Shearing strength of lift slabs had interested the authors for several years and they welcomed the opportunity provided by the Division of Architecture to investigate this problem. The authors feel that both structural engineers and research engineers in the field of structures have the same ultimate objectives in mind with respect to a research program. Research should be aimed at giving us a better understanding of the factors influencing structural behavior and strength, thus leading to better and more economical designs.

Mr. Brownfield's definition of failure "... a test structure has failed when it becomes useless or hazardous" does not agree with the usually accepted definition of failure, although it is correct from the point of view of serviceability. The ACI-ASCE joint committee on ultimate strength design* based their equations and ultimate load factors on tests which measured the ultimate load capacity of many specimens. The point at which a structure becomes useless or hazardous is subject to individual interpretation, whereas the ultimate load capacity can be impartially measured. When the load factors for design are selected, the behavior of the structure under all probable loads must be considered along with factor of safety against complete collapse.

Mr. Brownfield says "It is easy to propose a testing program...". Obtaining the maximum amount of new useful information within the limit of available time and funds is not so simple. The authors felt that basic in-

*ACI-ASCE Committee 327, "Ultimate Strength Design," ACI JOURNAL, V. 27, No. 5, Jan. 1956 (*Proceedings* V. 52), pp. 505-524. See also Paper No. 809, *Proceedings*, ASCE, V. 81, Oct. 1955.

formation on the behavior of lift slabs in shear utilizing a wide range of variables was desirable. They also felt that tests in the rather limited range of past and present lift slab designs would not give sufficient information regarding the effect of the various variables on the shearing strength of these slabs. The specimens were chosen to investigate the effect of amount of prestress, concrete strength, slab thickness, collar size, and recessing of the collars upon the behavior and ultimate strength.

The test arrangement of a square slab simply supported on all four sides was selected for two reasons. First, the authors agreed that it was impossible to simulate actual conditions, but decided that the selected test arrangement fulfilled the major requirement of similar moment condition at the lift-slab collar. Second, the size and shape of the specimens agreed with the reinforced concrete slabs tested by Elstner and Hognestad[2] so that the shearing strength of prestressed and reinforced concrete slabs with lift-slab collars could be compared with reinforced concrete slabs with concrete columns.

A circular specimen, as Mr. Brownfield suggests, was considered. The slight advantage it might have had was ruled out on basis of cost in an attempt to get the maximum number of specimens with the funds available.

Larger specimens were also considered and rejected for two reasons. First, it would have been almost impossible to make the larger specimens fail in shear before flexure failure occurred. Mr. Brownfield will note that many of the load-deflection curves reported by the authors show that the failures are predominately of a flexural nature. Second, although a special loading frame could have been constructed to accommodate larger specimens, it was desirable to use the available 4,000,000-lb testing machine (one of the world's largest) the bed of which limited the size to about that used.

The authors recognize that other problems such as the effect of large holes adjacent to the collars and the shearing strength of the slabs at exterior columns require additional research. It is hoped that the investigation reported in the present paper has been a first step in the solution of this interesting and complex problem.

Disc. 55-33
(Also 55-5, 55-16, and 55-22)

Discussion of a four-part series by James E. Backstrom, Richard W. Burrows, Harry L. Flack, Richard C. Mielenz, and Vladimir E. Wolkodoff:

Origin, Evolution, and Effects of the Air Void System in Concrete*

By LEVI S. BROWN, M. SPINDEL, BIRGER WARRIS, and AUTHORS

By LEVI S. BROWN†

Personal experience now totalling a mile or more of linear traverse air measurement in concrete permits the venture of a few comments, mostly by way of answer to three or four questions commonly raised.

1. How do you distinguish entrained air from entrapped air? You don't. The survey represents a summation of all void space observed. For one thing, there is no secure way to establish a size range distinctive between the one and the other. Even if a critical size were established, the true diameter of a void is not always perceivable on a random section. For another thing, measurement of air in hardened concrete in practical effect commonly is made as a check against air as determined by pressure meter in the fresh concrete. Air indicated by the pressure meter is a summation of all void space. The pressure meter does not distinguish between entrained and entrapped air.

2. How do you distinguish water voids from air voids? This is a question frequently asked. Water voids, readily recognizable as such, are rarely encountered in good concretes. Where they appear, they practically always are in the form of thin, shell-like openings resulting from subaggregate bleeding. They are readily recognizable as such, and in the traverse are ignored.

3. Why not work with photomicrographs, with magnifications possibly secured or increased by enlargement? That has a dual answer. For one thing, it is almost impossible to secure an operable print of an area of concrete sufficient for adequate statistical representation. Also, perfection of surface preparation still is required, as well as visual scanning of the surface to locate a suitable field. Another matter concerns a feature less generally known. A photomicrograph represents essentially a monocular view directed normal to the surface. It is disconcerting to discover how ineffectual it can be to perceive some of the voids in the surface by this approach. That holds even for direct visual observation, let alone trying to make appropriate distinction on the photographic print.

*ACI JOURNAL, V. 30 (*Proceedings* V. 55): No. 1, July 1958, p. 95; No. 2, Aug. 1958, p. 261; No. 3, Sept. 1958, p. 359; No. 4, Oct. 1958, p. 507. Disc. 55-33 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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Such uncertainty is reduced substantially to zero in surface scanning by stereoscopic binocular with illumination from one direction only and incident at a low angle, around 30 deg from the horizontal. With this arrangement, and a suitably prepared surface, it is possible to come pretty close to the impossible in ready recognition of air voids, not only as they are intrinsically small, and they can be perceived down to 15 microns or less, but also in the more difficult aspect of smallness by virtue of sectioning close to the bottom of the void. Perfection in perception of the latter is impossible. That is of little moment with respect to total air, but it is a matter of perhaps critical significance in studies of size and distribution such as presented by these authors.

4. It is well to visualize the Rosiwal method in the more general approach offered by Rosiwal. Suppose one could be a Maxwell demon, of a sort that could roam completely at will in any direction, looped, curved, or otherwise, within the confines of a heterogeneous solid. Separate summation of distances pursued in the several components, as related to the total distance, suffices to determine the volume proportion of each, assuming a statistical sufficiency in total length. Physically, a plane section offers the only easy entry to the interior. Here again the path traced on the plane can be any sort of devious route, but practical application is most convenient by straight line. Since a continuous straight line of sufficient length is impracticable, the integration is made along a series of parallel straight lines across the available specimen, the arbitrary spacing being such as to afford a sufficient total length and representative covering of the plane.

As noted by the authors, a total traverse length of 8 to 10 ft is usually sufficient with concretes of 1½-in. maximum size aggregate. Experience has shown that this length is not always enough for secure statistical representation of entrapped air. In the absence of entrapped air, an even shorter total length may be sufficient.

All of this is by way of emphasizing the niceties required for accurate integrations. This is really a precision operation. The random plane must be a plane, and the random line must be a line, in the mathematical sense. Preparation of such a plane on a concrete surface requires meticulous methodology, and settings on void intercepts, numbering in the hundreds for a single integration, along the line must be precise. Otherwise, the eventual summations are without merit.

5. All of which, in turn, is a way of emphasizing for the benefit of the reader one feature that modesty forbids to the authors. The reader should be made aware of the patient and painstaking care that has gone into acquisition of a literally prodigious volume of observational data here presented. One may agree with these authors in the assertion that much still remains to be done, but that in no way lessens the merit of high compliment for what they have accomplished.

By M. SPINDEL*

This discussion of Part 3 of the series refers also to Parts 1 and 2.

The writer will not deal with the many important details given in these papers but intends to give a general survey of what has already been achieved with air-entrained concrete in theory and practice since it started in the United States and what is still left to be done, to which the authors of this paper certainly are competent to contribute a great deal.

The influence of air voids on the properties of structural concrete have been dealt with by many research engineers, such as Feret, Talbot, Richart, and the writer. These investigations showed without doubt that air voids reduce strengths and other important qualities of concrete.

When the beneficial effect of air-entrained concrete was discovered in the United States, it at first seemed to be the idea on some jobs that the more air entrained the better. Certainly at the same time as other concrete engineers in the U. S. did, the writer in a discussion of the paper "Function of Entrained Air in Concrete" pointed out [ACI JOURNAL, V. 14, No. 3, Nov. Supplement 1943 (*Proceedings* V. 39), p. 544-1] that the air-cement ratio was of the same importance as the water-cement ratio and, therefore, both had to be limited and checked in the same way. This was soon solved in a satisfactory way by limiting the volume of air entrained in concrete to 3-6 percent of the volume of hardened concrete.

Another question pointed out by the writer at the beginning was that it did not suffice to limit the percentage of air but that also "shape, size, and distribution of air pores had to be determined" (*Transactions*, Society of Engineers, London, Dec. 1945). In that paper by the writer and R. T. Quinn, it was pointed out that "shape, size, and distribution of air voids should be tested by microscopic examinations and also by appropriate absorption tests, the latter giving the so called 'saturation coefficient,' which indicated the resistance to frost of concretes and natural stones alike." This question was first dealt with in the U. S. by carrying out microscopical examinations with regard to air voids partly according to old methods and partly according to rather new methods to which work the authors referred in their paper and some of which they used in their own research work.

Although everybody concerned will agree that immense work has been done —first in the U. S. and later in Europe—to find out which improvements could be obtained by air entrainment, which agents would do best, and how these agents have to be used to get the most satisfactory results, the problem of microscopical examination of concrete has not yet been solved, not even as far as it has in the testing of natural stones.

In the writer's opinion the properties of natural stones and concrete have to be compared from time to time. Some concrete engineers might be interested to learn that as early as 1914, with the then first manufactured rapid hardening portland cement, high quality concrete was made which surpassed

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the strength, impermeability to water, and resistance to weather and frost of most of the readily available natural stones. Soon after World War I the first large European dams to use concrete without any natural stone facing were built and these dams withstood completely the water pressure and the weather and frost at an altitude of more than 6000 ft without any loss of impermeability or resistance to weather and frost at this high altitude.

Nevertheless, the writer, who was responsible for the design and making of this rapid hardening portland cement and the concrete used, realized the difficulties which might occur later because even the best concrete could not be considered to last forever and that sooner or later it might start deteriorating in the same way as do most of the natural stones exposed to like conditions.

Therefore, in the writer's opinion, concretes subjected to severe exposure conditions have to be tested in a manner similar to those suggested by the special Committee for Testing the Weathering Qualities of Natural Stones. At the Congress of the International Association for Testing Materials in New York in 1912 a paper was submitted with suggestions for carrying out the necessary microscopical examination which was not accepted; nor was the proposal for determining the water saturation coefficient accepted.

In the writer's opinion, the greatest progress in testing concrete will be achieved with the help of the microscope, as soon as the difficulties of examining concrete in this way are removed. These lie, first, in preparing the specimens of concrete containing air bubbles. If treated like metals the air pores as a rule are partly demolished and partly filled in with dirt during grinding and polishing or filled in deliberately with a red paste to be seen better. It is necessary to differentiate between ordinary holes and entrained air bubbles since only the latter are useful. Further, it is necessary to find out which holes of broken concrete are water-repelling and which water-absorbing. The writer is carrying out such microscopical tests with special objectives for dark ground illumination on broken pieces without any grinding, polishing, or etching and thus examines the whole inner surface of the holes of air-entrained concrete or voids. The results will be published as soon as satisfactory photos can be made.

Although much useful information was given in the papers in question there was only one photo in the four parts of the paper showing the real structure of the concrete investigated, and therefore some users will not benefit as much as they were expected to do by the work done by the authors.

As to the water saturation coefficient, the writer is of the opinion that it would still improve the excellent results obtained with many hundreds of freezing and thawing cycles carried out at present in all well-equipped testing laboratories.

The idea of this discussion is to appreciate the achievements with entrained air, especially if combined with some dispersion of cement as often done recently and to point out that, although good concrete is equal to the best natural stones with much superiority in use, the methods of testing have still to be perfected.

By BIRGER WARRIS*

The authors are to be complimented on their extensive study of the air void system in concrete. Considering the time-consuming task of measuring these air voids, the large amount of collected data is impressive.

The Swedish Cement and Concrete Research Institute began investigations on the air void system in concrete by means of the linear traverse method in 1956. Measurements on air void systems in cement paste had been started earlier.† The measuring method has been the main object of most of the studies on concrete carried out until now, but results on the influence of different factors upon the air void system have also been obtained.

Before the investigation was started the linear traverse table was calibrated by means of specimens borrowed from the Portland Cement Association in Chicago, where we had previously received a description of the apparatus. Good agreement was found between PCA's results and ours. We found, however, that air void parameters measured on specimens prepared and ground by us differed appreciably from the parameters determined on the American concrete. By our grinding method, which followed a method used here in geological laboratories, water was applied, and thinking that this might cause solution of the cement we changed to kerosene. Table A gives the results; the first three specimens are from air-entrained concrete, the last three contain no air-entraining agent (air contents of this amount are normal for concrete here). The confidence interval has been calculated on 90 percent level.

The values of α and $\bar{L}$ for the same specimen but for different grinding methods show fairly large discrepancies, especially for non-air-entrained Specimens 57101, 57111, and 57121. The high values of α gave rise to a detailed study of the void distribution. A camera was coupled to the switch which is depressed when a bubble passes the cross hair of the microscope. When the bubble "leaves" the cross hair, one exposure is made of the counter, which measures the traverse through air bubbles. Every bubble chord is thus reported and can be calculated from the film tape. By this analysis it has been found that the differences in α were due to a high number of chords below 50 microns for the specimens ground with water. This experience may give an

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†Danielsson, Ulf, and Wastesson, Anders, "The Frost Resistance of Cement Paste as Influenced by Surface Active Agents," *Proceedings* No. 30, Swedish Cement and Concrete Research Institute, Stockholm, 1958.

TABLE A

Specimen No.	Grinding with water			Grinding with kerosene		
	Air content, percent	Specific surface α, in.$^{-1}$	Spacing factor $\bar{L}$, in.	Air content, percent	Specific surface α, in.$^{-1}$	Spacing factor $\bar{L}$, in.
57071	4.7 ± 0.9	380 ± 80	0.013 ± 0.002	4.6 ± 0.8	430 ± 100	0.012 ± 0.002
57083	4.9 ± 0.8	580 ± 100	0.009 ± 0.001	5.0 ± 1.0	380 ± 100	0.012 ± 0.002
57091	4.0 ± 0.6	790 ± 130	0.007 ± 0.001	3.3 ± 0.7	610 ± 130	0.010 ± 0.002
57101	3.8 ± 0.8	530 ± 130	0.010 ± 0.002	3.9 ± 0.9	380 ± 100	0.014 ± 0.002
57111	3.7 ± 0.8	580 ± 130	0.009 ± 0.001	3.5 ± 0.9	330 ± 100	0.017 ± 0.003
57121	2.4 ± 0.6	760 ± 200	0.009 ± 0.002	1.8 ± 0.6	640 ± 200	0.012 ± 0.002

impression of the delicateness of the linear traverse method. The writer would be interested to know if the authors have had troubles of the same type.

Fig. A-1 gives, according to the writer's opinion, a rather optimistic impression of the accurateness of the air content estimation. The operators here usually report a "standard error" of about 3 to 5 percent air per inch traverse,[12] which means that the standard error is about 0.3-0.5 percent air when the traverse length is 100 in. For the spacing factor the values are 0.005-0.02 in. per in. traverse, giving a standard error of 0.0005-0.002 in. for a 100-in. traverse. Table B, giving results from a specimen measured on two sides, shows that by short traverses fairly large divergencies can be obtained between the average values of air content and spacing factor. For economical reasons it has been necessary for us to limit the traverse length to about 2 m (about 80 in.). The table gives 90 percent confidence limits.

Our experiments with varying vibration time are not in concordance with the authors'. Using prisms 3 x 3 x 24 in. (horizontally cast) and varying the time in which the prisms are placed on the vibrating table we found the results in Table C.

In this case $\bar{L}$ decreased when the vibration was prolonged to 180 sec. A similar effect has been observed with non-air-entrained concrete. The divergence between these results and those reported by the authors may be due to great differences in the consistency. The concrete referred to in Table C was very stiff compared with usual American concrete (5 deg of VB, or about ½-in. slump). We have found also that the mixing time influences the void distribution as can be seen in Table A. The mixing times for Specimens No. 57071, 57083, and 57091 were 300 sec, 90 sec, and 900 sec. When air content and spacing factor are reduced, the specific surface is increased.

The series presents an enormous compilation of data on the air void characteristics which are of great interest. It has been encouraging to the writer to find that the tendencies brought out by our experiments—being far less extensive—are in accordance with the authors' results on most points. We have thus found the same influence of water-cement ratio and of temperature (cooled materials). An investigation of commonly used air-entraining admixtures has shown no significant differences between the air void properties.

The writer would like to finish his remarks with some discussion concerning the relationship between frost resistance and void-space ratio.

TABLE B

One side, every other traverse	One side	Both sides, every other traverse	Both sides
17-19 in. traverse	36 in. traverse	33-39 in. traverse	72 in. traverse
$A = 4.8 \pm 1.4$ $\bar{L} = 0.20 \pm 0.06$	$A = 3.6 \pm 1.0$ $\bar{L} = 0.20 \pm 0.05$	$A = 4.6 \pm 1.0$ $\bar{L} = 0.20 \pm 0.04$	
$A = 2.4 \pm 1.1$ $\bar{L} = 0.19 \pm 0.08$			$A = 4.4 \pm 0.8$ $\bar{L} = 0.22 \pm 0.04$
$A = 4.4 \pm 1.9$ $\bar{L} = 0.21 \pm 0.06$	$A = 5.2 \pm 1.2$ $\bar{L} = 0.25 \pm 0.05$	$A = 4.2 \pm 1.3$ $\bar{L} = 0.26 \pm 0.07$	
$A = 6.0 \pm 1.6$ $\bar{L} = 0.30 \pm 0.09$			

TABLE C

Specimen No.	Vibration time, sec	Air content, A, percent	Specific surface α, in.$^{-1}$	Spacing factor $\overline{L}$, in.
57011	30	4.2 ± 0.9	710 ± 180	0.008 ± 0.001
57013	180	3.0 ± 0.7	940 ± 280	0.006 ± 0.001
57015	1200	3.3 ± 0.6	990 ± 250	0.006 ± 0.001

Considering the practical point of view first, it seems evident that the spacing factor is a complicated parameter, involving an elaborate experimental procedure. It must therefore be encouraging to every concrete technologist concerned with frost resistant concrete to read the authors' conclusions in Part 4: "A satisfactory entrained air void system in job concrete is assured if 'Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)' is followed, provided the air-entraining admixture meets the requirements of ASTM C 260. . .". Even if some small divergences from this statement may be found later the question of how to make a frost resistant concrete seems to be settled for the practician. The problem is solved by specifying an air content and a satisfactory agent.

When considering basic research, however, much is left to do. Powers' hypothesis on hydraulic pressure and Taber-Collins-Pickett's hypothesis on microscopical ice segregation offer two possible explanations on the mechanism acting in concrete when freezing. Both hypotheses include the fact that the air void distribution (as well as some other factors as permeability and strength) is of importance to the frost resistance, and the authors' results must therefore be considered as a support to these hypotheses.

Our studies on air-entrained concrete show that when water-cement ratio is greater than about 0.5 (by weight) the frost resistance (freezing and thawing in water) in one way or another is connected with a parameter of the form $A/(p \times w/c)$ where A is air content in percent, p is paste content in percent, and w/c is water-cement ratio by weight. The specimens show simultaneous loss in weight and modulus of elasticity. Air void characteristics are not known. Because of the correlation between water-cement ratio and spacing factor found by the authors, these results are in accordance with their findings. The writer wonders, however, how the authors interpret the variation in frost resistance which they have found when water-cement ratio, and, thereby, spacing factor vary (Part 3, Fig. 1). Is this variation due only to the spacing factor or has the water-cement ratio still another influence, e.g., through strength or permeability? When specimens are frozen in air and thawed in $CaCl_2$ brine we find the weight loss to be better correlated to the air content than to the spacing factor. This seems difficult to explain by the two mentioned hypotheses. It appears to the writer that much research work is needed before we know how concrete is destroyed by frost, and whether the spacing factor is the *only* important parameter in frost resistance.

To sum up, the writer wishes to congratulate the authors for the work done. The knowledge on this topic has been considerably increased.

AUTHORS' CLOSURE

We are grateful to Messrs. Warris, Brown, and Spindel for their discussion of various parts of this series. Mr. Spindel has offered several interesting comments. We look forward to publication of the data and experience to which he refers.

Mr. Warris' discussion provides a considerable amount of data to supplement those in our papers. We have not explicitly evaluated the differences indicated in the parameters of the void system when different methods of preparation of the ground surfaces are employed, but it was readily apparent in our early work that the quality of the surface markedly influences the determined value of specific surface and spacing factor, primarily because rough surfaces destroy the smallest void sections, so tending to decrease the value of α and increase the value of $\overline{L}$. Void sections less than 50 microns in diameter are especially susceptible of obliteration.

Table A indicates that substitution of kerosene for water as the grinding medium consistently increases $\overline{L}$ and, for all specimens but one (that with minimum α for water grinding), decreased α. In spite of the fact that water would appear to be unsatisfactory as a grinding medium because of the possibility of softening and leaching the concrete, experience indicated to us that water was superior to both oil and ethylene glycol as a grinding medium primarily because of the difficulty involved in thorough washing of the surfaces to remove the abrasive material following successive stages of grinding. Such residual abrasive not only impairs the development of a plane, scratch-free surface, but also the particles clog and conceal small air voids. It is possible that the effect observed by Mr. Warris resulted at least in part from these phenomena.

In view of Mr. Warris' comment on Fig. A-1, the statistical analysis (Table D) is presented for the data from which that figure was constructed.

The data in Table D and Fig. A-1 were obtained by linear traverse on one surface of each specimen over an area of approximately 4 x 4 in. The traverses were spaced 0.08 in. apart. The concrete contained ¾-in. maximum size aggregate; water-cement ratio = 0.51 by weight; all specimens were well vibrated. The above statistical analysis assumes each four consecutive traverses to be one observation.

The results are not as "optimistic" as may appear from Fig. A-1.

Concerning Mr. Warris' discussion of the effect of vibration of concrete on the parameters of the void system, we did not evaluate the effect of external

TABLE D—STATISTICAL ANALYSIS

	Specimen A	Specimen B	Specimen C
Standard deviation	0.3174	0.6531	0.9790
Coefficient of variation, percent	30.5	29.0	20.4
Air content (9 chances out of 10), percent	1.04±0.19	2.34±0.40	4.79±0.60
Air content (99 chances out of 100), percent	1.04±0.40	2.34±0.71	4.79±1.06

vibration. We would not expect the effects to differ substantially from those obtained by internal vibration if other properties of the concrete mixtures are held constant. As Mr. Warris points out, the concrete for which data are presented in Table C is characterized by a slump considerably less than that for concrete included in our investigations. Our data have shown that the void system is influenced by the consistency of the concrete, if other qualities are essentially constant. Probably, the stiff consistency of the low-slump concrete prevented the great loss of air observed for the plastic concrete used in our tests (compare Mr. Warris Table C with Table 6, Part 3, of our report), the main action of the external vibration in Mr. Warris' tests being comminution of the bubbles.

Mr. Warris' observations on the effect of mixing time on the void system of concrete are valuable, particularly because they touch on a point not investigated by us. As is indicated above, these results probably cannot be used directly to predict results to be obtained with concrete of plastic consistency, such as would yield a slump of 2 to 5 in.

We agree entirely with Mr. Warris that basic research on freezing and thawing durability and scaling resistance of concrete should be continued. His observed direct relation between frost resistance of concrete and the ratio $A/(p \times w/c)$, for w/c greater than 0.5 seemingly relates to the magnitude of the freezable water content of concrete of moderate or high water-cement ratio. Under given conditions of moist curing or immersion in water, the freezable water content of the cement paste in concrete will increase as the volume of the cement paste and the water-cement ratio increase. With a low content of freezable water, for given spacing factor and under given conditions of freezing, the pressure developed within the concrete will depend primarily upon the permeability of the cement paste. At some higher content of freezable water, the pressure developed within the concrete will depend primarily upon the capacity of the air void system to accommodate the volume of water moving from the portions of the cement paste being frozen, that is to say, permeability and void spacing may be adequate, but the capacity of the air voids is insufficient.

Spacing factor is not the only control of freezing and thawing resistance of concrete. Mr. Warris calls the authors' attention to Fig. 1, Part 3, of our series of papers in which variations in indicated frost resistance accompany changes in both the water-cement ratio and $\overline{L}$, spacing factor. It has been established that other things being equal, the quality of concrete, such as is measured by strength and permeability, decreases with an increase in water-cement ratio. Also, there is an optimum air content and spacing factor for any given water-cement ratio.[20] Fig. B of Reference 20 shows the effect of increasing water-cement ratio on the freezing and thawing resistance of concrete. At air content about 7 to 10 percent, an increase of water-cement ratio from 0.45 to 0.55 by weight increases the expansion per cycle from 1.5 millionths to 5.0 millionths and the number of cycles necessary to cause a 25 percent weight loss for the specimens decreases from 1000 to 400. Un-

fortunately, we do not have the air void parameters of these concretes, but from similar concretes we would expect the spacing factor to be in the neighborhood of 0.004 to 0.006 in., a figure sufficiently small to provide good protection for the concretes. Since the cement, aggregate, and air-entraining agent used in these concretes were the same, the effect on frost resistance must be attributed to difference in water-cement ratio and the attendant physical changes, as in strength, permeability, etc., as Mr. Warris suggests.

We are pleased to have Dr. Brown's discussion as a supplement to our report. He leaves little for us to say in reference to the general aspects of air void traversing. Referring to the last sentence of his paragraph No. 2, we would like to note that our traverses include all air voids observed in the surfaces, including the occasional openings apparently resulting from bleeding and settlement of the fresh concrete. These are included in the analysis for two reasons: (1) to attempt to eliminate them introduces a subjective factor in the measurement and (2) such voids, once emptied of water by drying of the concrete or as a result of the curing and hydration process, act as air voids just as do "entrapped" air voids of equal surface area, even though both are relatively insignificant in their effect on frost or scaling resistance.

Dr. Brown and Mr. Spindel have noted the difficulties involved in photographing or otherwise reproducing precisely the microscopical structure of concrete. Actually, however, the authors and their colleagues in the Engineering Laboratories of the Bureau of Reclamation thoroughly investigated a wide variety of techniques for reproducing the details of the air void system as presented on finely ground, plane surfaces, as well as automatic (electronic) methods for making the linear traverse. Means of preparation of the ground surfaces included use of fluorescent dyes, pigments, inks, cast rubber molds, and impregnation by alpha-emitting polonium. None was satisfactory. Considerable advance was made in development of photographic methods by our colleague, William Y. Holland, particularly positive transparencies for electronic scanning, but it was found impossible to consistently reproduce the void structure of the surfaces, especially the void sections of dimensions approaching those of the grain of the photographic film. Also, it was found that the apparent dimensions of the void sections were modified significantly by such variables as lighting, exposure time, and details of the developing techniques.

T. F. Willis has kindly pointed out an error in our citation of Reference 18 on p. 513. The reference should be to Appendix II of a 1949 departmental bulletin of the Missouri State Highway Department by T. F. Willis. This publication is referred to in footnote 5, p. 205, of our Reference 8.

It is our hope that our report will contribute to more fundamental knowledge of the development of the air void system in concrete and that it may encourage basic research along several lines, such as the physical-chemical process by which surface-active agents influence the air void system; the influence of other types of admixtures upon the air void system; and the effect of ambient temperature, and of placing and finishing operations.

Discussion of a report by ACI Committee 614

Proposed Revision of Standard 614-42:

Recommended Practice for Measuring, Mixing, and Placing Concrete*

By EDWARD A. ABDUN-NUR, J. E. GRAY, E. L. HOWARD, HARRY F. IRWIN, BAILEY TREMPER, WALTER K. WAGNER, STANTON WALKER, BYRON P. WEINTZ, D. K. WOODIN, and COMMITTEE†

By EDWARD A. ABDUN-NUR‡

The committee is to be highly commended for a fine piece of work. The expressed approach of the committee, to assemble recommendations that produce a "high standard of practice" rather than "common practices," is realistic and progressive. Without progress, the human race would still be living in caves, and the committee's policy, to stimulate progress in terms of modern concrete know-how and technology, through a slow discretionary evolution without working hardship on anyone, is desirable and wise.

The recommendation for finish screening seems to have elicited unjustifiably exaggerated criticism. Two reasons appear to be behind this: (1) lack of familiarity with the purpose and need for this operation, and the benefits that result from its use; (2) fear that it will result in a large increase in the cost of producing concrete.

It must be stressed that the purpose of finish screening is *not* to readjust the gradation which has been carefully produced at the aggregate plant, but to eliminate the variation in undersize (resulting from degradation in handling, and dirt accumulated in stockpiling) from batch to batch, and to provide a final remixing of the particles found in the stockpile in various degrees of segregation, during the final handling. This precludes any further opportunity for segregation or degradation. The rejected fines can, if they meet specifications, be added to the fine aggregate, or be used for other purposes around the site, and are thus in most instances not wasted.

Asphalt plants have for years used finish screening to help control the uniformity of their mixes, and no one would think of operating such plants without final screening immediately before batching. Is not portland cement concrete as worth controlling properly as bituminous mixes?

*ACI JOURNAL, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 55), p. 535. Disc. 55-35 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 12, June 1959 (*Proceedings* V. 55).
†These discussions were presented or summarized at the 55th annual convention, Los Angeles, Feb. 24, 1959.
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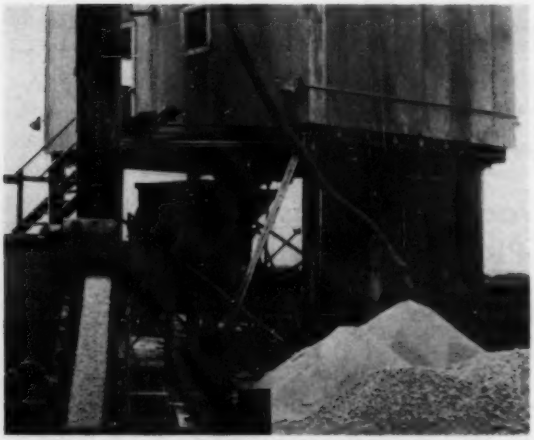

Fig. A—Mixing plant showing rejected fines being emitted from pipe at right

From the cost angle, a 1-deck, a 1½-deck, or a 2-deck screen is all that is needed to do the job, depending on the number of size separations required. In most instances, the cost of outfitting a new plant with finish screens runs from about $4000 to $8000. Existing plants may require minor modifications, but simple, low-cost solutions for these have been found in most instances. Maintenance on screens is minor, and operating costs are low. Considering the number of cubic yards of concrete that can be batched during the useful life of a finish screening setup, even a $10,000 capital outlay would raise the cubic yard cost so little that the increase would be lost in today's prevailing unit prices of finished concrete.

As a result of finish screening the cost of stockpiling is reduced, the cost of reclaiming the bottom of the stockpile by running the material back to

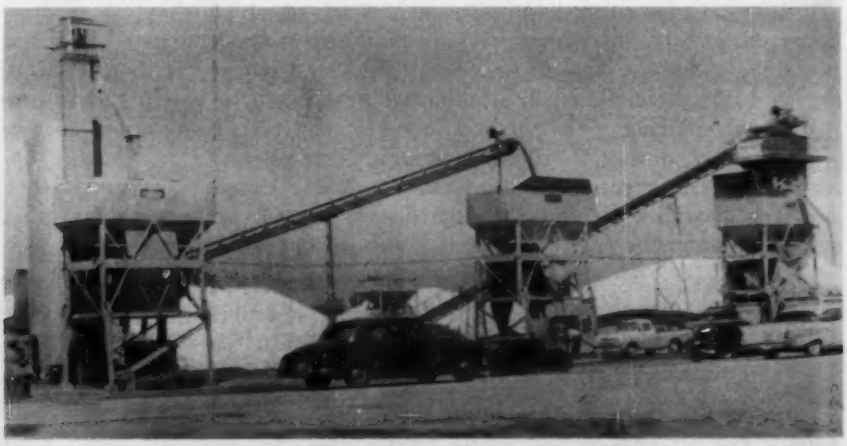

Fig. B—A three-stop batching operation employing finish screening. Note excess fines coming out of the chute on the right

and through the aggregate plant is eliminated, and the cost of delays due to dirty or segregated portions of the stockpile is done away with, so that the net result is usually a saving in favor of the finish screening. Under such circumstances, few plants can actually afford to be without finish screening.

With increasing use of strength specifications geared to uniformity, anything that will improve uniformity and thus reduce strength requirements will better the producer's profits. Under such specifications, finish screening, by eliminating the variation in fines from batch to batch, increases uniformity and thereby profits.

The value of any operation can best be judged by the results that it produces. On the Illinois Toll Highway ("Control of Concrete Mixes," by Edward A. Abdun-Nur and Joseph J. Waddell, ACI JOURNAL, Mar. 1959, p. 947), it was possible to reduce the cement factor on many concrete operations and still meet requirements because of the uniformity attained—part of this increased uniformity in these cases must be credited to finish screening. The accompanying photographs give a striking indication of what finish screening will do.

Fig. A shows a small prestressed concrete mixing plant with a stream of fines being discharged from the reject pipe on the right, and the aggregate feed going up to the finish screens on the left. A small reject pile of fines appears under the waste pipe. Had the fines been left in, the aggregate would undoubtedly have developed variations in water requirements from batch to batch that would have made good control impossible.

Fig. B and C provide a more striking example of such variation. Fig. B depicts an over-all three-stop batching operation. The coarse aggregate bins at the right were fitted with finish screens over the top, and one can readily see the stream of rejected fines coming out of the side chute. Fig. C taken a minute or so later to get a closer view of the aggregate operation, shows that the aggregate is still coming over the screens, but without any fines coming out of the reject chute—a definite indication that the excess fines in stockpile were not uniformly distributed. Here again, a good sized

Fig. C—Closer view of operation shown in Fig. B taken a few minutes later. Note that no fines are coming out of the reject chute, indicating that excess fines were not uniformly distributed

pile of reject material is in evidence. The two pictures represent random conditions, inasmuch as the difference in the reject chute was not noticed until the photographs had been developed.

The committee's recommendations were approved unanimously by all its members, by all members of the Technical Activities Committee, and by all the members of the Standards Committee. Under the circumstances, the various recommendations have had the careful study and scrutiny of a good cross section of the Institute.

Attention is called to the fact that the recommendations leave the use of finish screening entirely to the discretion of those charged with the individual project responsibility. No one is forced by these recommendations to use this procedure.

In fact, all the committee is doing, is to dangle progress in front of the engineers and architects who design, specify, and supervise concrete construction—and in the words of L. R. Howson, past president of the American Society of Civil Engineers, in a speech at the 1958 Portland Convention: "There is no appeal from the judgement of progress."

By J. E. GRAY*

The philosophy of the committee in preparing this report was that the recommendations should encompass the best practices known to obtain the best concrete possible, even though many of the recommendations are not in current use. While in theory this principle may be good, I believe that the committee has applied it beyond the dictates of experience n some respects and particularly with regard to aggregates, especially in view of the fact that "...the committee considers this brief statement on aggregate requirements an essential part of these recommendations... Due to segregation, breakage, and contamination during handling ... unless coarse aggregate is placed in batch plant bins directly from screening operations, finished screening at the batching plant is recommended." This implies that the concrete aggregates for normal structural and paving uses are not satisfactorily graded by producers of aggregates. Last year, Harold Allen, chairman of Subcommittee C-14 of ASTM Committee D-4 on Standard Sizes of Coarse Aggregate for Highway Construction, reported that a study of the specifications of the various state highway departments with respect to gradation requirements for aggregates for concretes were essentially in accord with Simplified Practice Recommendations R 163-48. The simplified practice recommendations have been in use for over 20 years. They have been reviewed by engineers and producers with the result that the size requirements provide aggregates of a gradation suitable for good quality, economical concrete. Actually, this latest review of the use of simplified practice sizes

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clearly implies that aggregates being supplied for highways from commercial sources are being properly graded.

The point could be made by the committee that no criticism is being made with respect to current practices, but that recommendations are being made as to what is believed to be a better practice. Two objections raised to this kind of reasoning are: (1) no data are submitted to show the necessity of this recommendation for finished screening; (2) in its ultimate application, the quality of the concrete produced may be quite inferior to that being obtained today under the present system of procuring aggregates from established commercial sources.

In my 30 years of experience with the National Crushed Stone Association, concrete which has been found to be in a serious state of disintegration has most frequently been produced with aggregates from temporary roadside operations. If the aggregates as commercially supplied are so poorly graded that the contractor has to rescreen the material as it goes into the bins, why should he not produce his own aggregates? In other words, the recommendation of this committee would have the effect of encouraging all contractors to produce their own aggregates from roadside operations. There is no objection to temporary or roadside production of aggregates per se. I have seen top quality crushed stone produced from portable roadside operations where the contractor was amply and properly equipped. However, in many roadside operations, an attempt is made to produce with a minimum of equipment, which is often coupled with inadequate inspection. From past experience, it would appear that any recommendations which would have the effect of encouraging roadside production of aggregates would result in lowering the quality of the aggregate and consequently the quality of the resulting concrete.

The committee recommends effective inspection. If effective inspection were used in the production of aggregates and its processing into concrete, there would be no problem of uniformity of the concrete. This I know to be true from experience. While on the other hand, the recommendation to install equipment whose use supposedly will produce a better quality of concrete has a tremendous appeal to some, if by so doing, equipment might serve in place of inspection. Herein lies the great fallacy of this recommendation.

The economic aspects of this problem deserve serious consideration. The mineral aggregate industries have been supplying aggregate with the smallest increase in price of any building material for many years. This has been brought about by modernization of plants with increased efficiency and standardization of sizes. In the past few years a number of plants have made a capital investment in their modernization program of $500,000 to $1,000,000. Some of these plants are putting into stockpiles, aggregates more closely sized than specification requirements and supplying specification sizes by feeding the proper combination of separate sizes on a blending belt as the material is being loaded for shipment. The blending belt is located

in a tunnel under the stockpiles and the aggregate is fed onto the belt in proper proportions by automatically controlled feeders which previously have been calibrated.

With the huge demand for aggregates today, the aggregates industry is doing a magnificent job of supplying tremendous quantities of properly graded aggregates complying with specifications for quality at an economical price. This industry deserves the support and aid of the concrete industry and should not be thwarted with recommendations which are probably quite applicable to the restricted field of large dam construction but not to the normal field of structural and pavement concrete.

By E. L. HOWARD*

We cannot believe that good handling and stockpiling practices can be relaxed with finish screening. Without controls careless handling surely follows when finish screening is used. For this reason some do's and don't's must be specified if finish screening is recommended. Some persons have suggested that "hot plants" are a favorable example of finish screening.

We have had to live with bituminous concrete for a long time. Uniformity of grading in hot plants is notable for its absence. Anytime a grading specification must be met an inspector is at the plant continuously sampling, testing, adjusting the feed, etc. With all he might do, his combined gradings never approach the close control line possible in an average concrete plant.

To emphasize the importance of do's and don't's, Table A shows actual gradings from an airfield job. Aggregate was unloaded by clamshell from railroad cars, trucked to the storage area, and fed through finish screens to bins. The 18 percent increase in weigh-hopper undersize was due entirely to combined feed and inadequate screening following careless handling methods. Each piece of coarse aggregate was coated with wet sand.

Some causes of "dirty" gravel at ready-mixed concrete plants are easily seen. The most common is spillage and over-run of sand into gravel bins (rescreening not of help here). Another common cause is the loading of coarse aggregate in unclean hoppers and bucket lines. Breakage, due to handling coarse aggregate with loaders and bulldozers, causes undersize and fine rock dust coatings on the coarse aggregate.

The full extent of these causes and the resulting problems in concrete control has been evaluated from "guesstimation" far more than from actual test data. The test program here reported, was begun to gather data on the extent and cause of contamination.

In California the "cleanness" of aggregate going into concrete is measured by the cleanness value (CV) test. To make the CV test a specified amount of aggregate is washed by agitation in water and that portion which will

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pour through a 200 mesh sieve is added to the standard sand equivalent solution in a sand equivalent graduate to determine the cleanness value. A minimum value of 75 is specified.

As long as aggregates are rinsed free of minus 200 mesh material at the gravel plant, there is no difficulty meeting these specifications when the aggregate is shipped. The 14 batch plants studied are typical of ready-mixed concrete plants in northern California.

Coarse aggregate was sampled at the gravel plant rinsing screens, and at the batch plants as the trucks dumped, and at the weigh-hoppers. Tables B, C, and D show the loss in cleanness values when the aggregate was handled. Materials shown in Table B were handled in the normal manner at the batch plant. Before the tests reported in Table C were made, all handling equipment was washed clean. The reduction in CV from 75 to 55 is due entirely to the minus 200 mesh material rubbed loose from the aggregate particles.

Batch Plant E receives its aggregate directly from the rinsing screens by belt conveyor. The abrasion due to truck or car loading and hauling or stockpiling is eliminated. The ¾-¼-in. gravel material shows no serious CV loss, (Table D). However, the abrasion as the gravel moved in the batch plant bins reduced the CV of the 1½-¾-in. gravel from 84 to 56.

At four locations sand had carelessly been allowed to overflow into the gravel bins. When this sand was found in the ¾-¼-in. gravel at Yard 44 (Table E), the CV was not affected. At this yard a "clean" sand is used. At Yard 81 only 4 percent sand caused CV failure of the ¾-¼-in. gravel.

TABLE A—INCREASED UNDERSIZE DUE TO FINISH SCREENING METHODS

US sieve	RR car	Batch plant storage	Weigh-hopper
2	100	100	100
1½	90	94	97
1	35	41	50
¾	6	16	24

TABLE B—COMPARATIVE CLEANNESS VALUES OF NILES AREA AGGREGATE

Sample taken at	CV, 1½-¾-in. washed gravel	CV, ¾-¼-in. washed gravel
Plant stockpile	84	86
Truck at batch plant	69	81
Weigh-hopper, batch plant	48	84

TABLE C—COMPARATIVE CLEANNESS VALUES OF LIVERMORE VALLEY AGGREGATE

Sample taken at	CV	Aggregate size (washed gravel)
Truck on arrival at plant	75	1½-¾ in.
Top of bucket elevator	58	1½-¾ in.
Weigh-hopper	55	1½-¾ in.

Note: Bucket line, bunker, and weigh-hopper were washed clean before these test loadings were made.

TABLE D—COMPARATIVE CLEANNESS VALUES OF AGGREGATE SAMPLED AT BATCH PLANT E*

Rinsing screen	Batch plant weigh-hopper	CV
1½-¾ in.		84
¾-¼ in.		89
	1½-¾ in.	56
	¾-¼ in.	84

*This plant fed directly from gravel mill by belt conveyor.

Here the sand has low sand equivalent values. The undersize at Yard 60 using American River aggregate, was rock breakage and some sand. This was the only location where the low CV could be attributed to contamination and breakage. Typically, the breakage was only in the 1½-¾-in. gravel. The inherent characteristic of Bay Area 1½-¾-in. gravel, viz, low resistance to abrasion, high adsorption for the clay laden water, etc., are the principle causes of the low CV's at San Francisco Bay area concrete batching yards.

Undersize, caused by breakage and handling between gravel mill and batch plant, was found only in the large gravel. Coarse aggregates that become contaminated with surface coatings from careless ground storage practices, etc., fail to meet the cleanness value requirements. Aggregate larger than ¾-in. are affected most because of the surface area differences.

About 90 percent of the coarse aggregates used in the 80 batch plants we service is ¾-¼ in. As noted above there is no "significant" undersize in this material. Except for the occasional project using large amounts of plus 1-in. aggregate, ready-mixed concrete plants in our area have no real "undersize" problem.

TABLE E—CV TEST VALUES AND "SIGNIFICANT" UNDERSIZE DATA OF SAMPLES TAKEN FROM BATCH PLANT WEIGH-HOPPERS

Yard No.	Aggregate size in.	Percent undersize	CV	Source	Batch plant loaded by:		
					Elevator	Belt	Clam
41	1½-¾	2	58	Livermore Valley	Rock only		
41	¾-¼	2	68	Livermore Valley	Rock only		
44	1½-¾	12(Sand)	61	Livermore Valley			X
44	¾-¼	10(Sand	94	Niles area			X
45	1½-¾	2	00	Livermore Valley		X	
45	¾-¼	2	81	Livermore Valley		X	
46	1½-¾	3	56	Livermore Valley		X	
46	¾-¼	2	81	Livermore Valley		X	
52	1½-¾	2	13	Niles area		X	
52	¾-¼	3	77	Niles area		X	
53	1½-¾	6	33	Niles area		X	
53	¾-¼	1	77	Niles area		X	
54	1½-¾	1	53	Livermore Valley	X-2 cu yd		
54	¾-¼	–	84	Niles area	X-2 cu yd		
55	1½-¾	9(Sand)	44	Niles area		X	
55	¾-¼	1	84	Niles area		X	
58	1½-¾	5	13	Livermore Valley		X	
58	¾-¼	2	80	Livermore Valley		X	
81	1½-¾	3	23	Livermore Valley	X		
81	¾-¼	4(Sand)	57	Livermore Valley	X		
82	1½-¾	1	29	Niles area		X	
82	¾-¼	1	81	Niles area		X	
60	1½-¾	8	68	American River		X	
60	¾-¼	1	86	American River		X	
68	1½-¾	0.5	81	San Joaquin	X		
68	¾-¼	1	89	San Joaquin	X		
69	1½-1	10(Sand)	94	San Joaquin	X		
69	1¼	1	94	San Joaquin	X		

HARRY F. IRWIN*

We are aggregate and ready-mixed concrete producers. As such we are interested in this report and feel that much of it is excellent information and that it will be widely used, and for that reason I would like to point out some features that I think contain questionable recommendations.

My interpretation of the report indicates that, regardless of qualifying clauses in the report which might be difficult to define in the field, finish screening is going to be required, and our experience with aggregates, which include sand, gravel, crushed stone, and slag does not indicate that such finish screening is required.

The aggregates, if properly handled, and we take every precaution to make sure they are properly handled, will not require this finish screening.

The proof of this is that the qualifications for the aggregates must be met at the batching floor before it goes into the concrete. What happens to the aggregate before that is not important except that it makes it easier to control at that point; but the control must be at the batching level and before it goes into the concrete.

If we can secure control at that point, the use of finish screening would not be justified, and finish screening in a plant in a metropolitan area would present problems that are almost insurmountable. We have zoning. We have the problem of space, which is at a premium in some metropolitan areas, and there are innumerable things that have made finish screening impractical and expensive.

I feel that recommending finish screening would result in higher cost and would not justify changing the methods now used in producing quality concrete.

By BAILEY TREMPER†

I would like to speak a little more about this question of finish screening from the standpoint of the engineer who has to deal with a large number of rather small jobs; small compared to a dam, but going all the time.

Suppose we do have aggregates furnished to a batch plant that meets the requirements of a standard size. There is no problem in handling them if each batch as it is mixed contains the material in those sizes.

It is a difficult problem for an inspector to determine that each batch that enters the mixer is within those limits. I do not think I need to say that we all know of cases where undersize does get into certain batches and that they do cause trouble.

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Suppose an inspector is trying to eliminate those batches. He gets a sample of the material in the bin, makes a screen test, and 10 or 15 min later he finds that it is outside of specification limits.

Suppose it is a job of 200 cu yd that is to be in one placement and 30 or 40 cu yd have already been placed. Is the inspector to shut the work down? What is he going to do with the portion that has been placed? The answer is, nine times out of ten, he will be obliged to let that placement go on and tell the producer that the next job has got to be better, and undoubtedly there will be attempts made.

And maybe you go along all right and in a few days something else happens and a bunch of fractured materials hang in the bin and come out in gobs in a few batches. It gets to be an almost impossible administrative matter to control.

Finish screening will be a long step in the right direction. The recommendation, as I see it, doesn't say this is mandatory. It says, it is good practice, and for my part, I think it is good practice and should be recognized as such; and such a recommendation has been proposed.

By WALTER K. WAGNER*

The Committee is to be commended for its work in pointing out those production methods that are today considered essential to quality concrete. The separate section on aggregates, however, attaches a more than significant importance to the size separation of coarse aggregates and effects of undersize material on the uniformity of concrete.

Finish screening at the batch plant appears to be the only recommended method for eliminating undersize, and this, preferably, over the batch plant bins. Even though many batch plants are structurally able to handle the screening setup topside, it could mean major modifications in the conveying methods of most present plants. Also, bins or hoppers for receiving the rejected undersize could impose problems of space or weight.

It is stated that "coarse aggregate can be rewashed, if necessary, during finish screening," and that "the water remaining on the material will not affect uniformity appreciably, . . ." I cannot agree with that statement based on experience with freshly washed aggregates, and I do not believe that erratic surface water on coarse aggregates could be a much "lesser evil" than a reasonable amount of undersize. For years certain cold weather batching has been acceptable practice when properly drained aggregates are used. But a requirement of rinsing or washing immediately prior to batching could virtually close up a plant during freezing periods.

If rinsing is not essential during finish screening then that operation could be responsible for creating a dust nuisance. Many plants in the United

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States today have become targets for certain zoning restrictions and need no further causes to jeopardize their present situations.

The report of the committee seems to imply that aggregates can not be produced and delivered to the batch plants with uniformity of moisture and grading. This might be quite true in the case of certain small aggregate producers, but those who are supplying the larger concrete plants are becoming increasingly aware of the need for greater uniformity in their products. Indeed, the terms "quality control" and "aggregate beneficiation" are noted quite often on convention programs of the National Sand and Gravel Association and the National Crushed Stone Association.

These producer interests coupled with those of the specification-writing groups such as ASTM and AASHO should be able to assure adequate quality of aggregates in keeping with improved production and handling methods. It certainly should not be necessary for ACI to implement regimentation which can be damaging to the industry at this time.

By STANTON WALKER*

I wish that I could feel that certain important parts of this proposed recommended practice were as good as most of it. I have read it with great care. Further, although not a member of the committee writing it, I have been in close touch throughout its development and I was accorded full opportunity to make recommendations concerning it. Some of my recommendations were accepted; those to which I attach most importance were not and they are the subject of this discussion. While there are several parts of the report which I might discuss, I shall confine myself to those dealing with aggregates, particularly, "finish screening" and "significant undersize."

Throughout the section on aggregates, there is the strong recommendation that all batching plants be equipped with screens so that coarse aggregates, however handled or produced, will be rescreened and rinsed as they go into the batch plant bin, unless they go there *directly* from the original screening operation. There is included a statement apparently considered as an escape clause but it is buried in the middle of one of the many paragraphs urging the general use of "finish screening." It reads: "But the main thing is the result, and if it *can be assured* by other means, there is no objection to them." (The italics are those of the committee, not mine.) That statement is equivocal, to say the least; it is completely impracticable of administration.

The economic implications of a screening plant on every established ready-mixed concrete plant, as well as other batching plants, whether needed or not, should be clear. Further, such screening plants would appear pretty much to destroy the value of the elaborate screening, handling, and blending equipment which are the rule in modern established plants for the production of mineral aggregate.

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TABLE F—EFFECT OF FINE SIZES OF GRAVEL ON STRENGTH OF CONCRETE*

Percent of fine gravel		Compressive strength		Modulus of rupture	
No. 8 to No. 4	No. 4 to ⅜ in.	lb per sq in.	Strength ratio, percent	lb per sq in.	Strength ratio, percent
0	0	2720	101	500	98
0	10	2850	106	490	96
0†	20†	2700†	100†	510†	100†
0	30	2700	100	490	96
10	0	2700	100	520	102
20	0	2660	99	505	99
10	20	2570	95	500	98
20	20	2580	96	485	95
10	30	2490	92	470	92
20	30	2400	89	470	92

*Data from NSGA *Circular* No. 7 (out of print), representing concretes made with gravel graded to 1½ in. maximum size, with portion coarser than ⅜ in. being equal percentages of ⅜-¾ in. and ¾-1½ in. Values represent cement factor of 5 sacks per cu yd, interpolated from tables based on the then-current usage of fixed proportions such as 1:2:4, etc. Ratio volume of mortar to volume of voids in coarse aggregate: 2.25.

†Well-graded coarse aggregate with no undersize, conforming to ASTM C 33, taken as 100 percent strength level.

The recommendation for finish screening appears to be based on the assumption that aggregates almost never can be delivered to the batching plant suitably graded unless such processing is employed—and, also on grading requirements more rigid than provided by such specification writing bodies as ASTM, AASHO, state highway departments, departments of the federal government, and others. Standard specifications fix tolerances on undersize which take into account the protection of the consumer and the economics of production, but the committee does not accept them. The committee says that the "significant" undersize is that smaller than about 4/5 to 5/6 of the designated minimum size and that such smaller sizes should be limited to not more than 2 or 3 percent. That means, for example, that an aggregate graded ¾-1½ in. would not be permitted to have more than 3 percent finer than about ⅝ in., the next smaller standard size sieve. This is to be contrasted with the general limitation of 5 percent finer than ⅜ in., for this size, permitted by accepted specifications.

No data are cited in support of this rigid limitation. It is true that a reference is given to the paper "Developments in Methods of Testing and Specifying Coarse Aggregates," by L. H. Tuthill.[9] That paper is silent on the quality of concrete or variations therein. It only cites the performance of the screening plants on two large dams. Conclusions regarding undersize appear to have been based on results achievable under those particular job conditions rather than on an evaluation of concrete quality or variability.

When I have asked the committee for data I have been shown pictures of badly segregated and contaminated stockpiles. These are not pertinent to my discussion. I agree that such aggregate stocks should be rescreened and, if necessary, rinsed before use. But it does not follow that this rescreening need be done on the top of a batching plant nor that there should be a screening plant on the top of each batching plant.

So far as my researches into the literature show, data bearing specifically on the point under discussion are meager. There are some tests which show strength relationships, and in a letter to the committee, I submitted certain

data along these lines from our own investigations. One was a series of tests carried out in 1927 in a determination of the effect of increasing amounts of smaller sizes of gravel on strength. The effect from amounts of undersize considerably in excess of conventional specification limits was not significant. These data are shown in Table F. At the same time I submitted results of tests on a gravel which, in the stockpile, looked dirty because of dust from fracture. Results of those tests are shown in Table G.

The committee might well argue that these tests and other similar ones did not furnish information on the difficult-to-measure property of workability, or on appearance, or on durability. I suggested to the committee that the cooperation of a number of laboratories be secured in the conduct of a carefully outlined series of tests to study this problem. I said that I felt that the participation of such organizations as the U. S. Bureau of Reclamation, the U. S. Army Corps of Engineers, and the Bureau of Public Roads probably could be secured. I said that we would be eager to take part in such a study and that I was sure that the same could be said of the National Crushed Stone Association and the National Slag Association. The chairman replied, "...I have been unable to convince myself that tests of concrete in which widely varying quantities of dirt and small undersize, such as would

TABLE G—CHARACTERISTICS OF FRESH CONCRETE AND RESULTS OF STRENGTH TESTS (SERIES J-108)*

Round	Cement, Sacks per cu yd	Water		Slump, in.	Air, percent (grav.)	Compressive Strength, psi	
		Gal. per sack	Gal. per cu yd			7-day	28-day
1—Gravel washed in laboratory (no minus No. 4 Sizes)							
A	6.49	5.03	32.6	4.3	4.9	2355	3305
B	6.52	4.88	31.8	3.4	5.1	2570	3680
C	6.58	4.83	31.8	3.2	4.3	2805	3850
D	6.62	4.80	31.8	2.7	3.7	2970	4065
E	6.61	4.81	31.8	3.0	3.9	2775	3955
Average	6.56	4.87	32.0	3.3	4.4	2695	3771
2—Same as No. 1; dust of fracture added with sand							
A	6.51	5.17	33.7	3.0	4.3	2480	3590
B	6.49	5.18	33.6	3.9	4.5	2435	3340
C	6.51	5.07	33.0	2.6	4.7	2745	3880
D	6.49	5.09	33.0	3.5	4.8	2540	3700
E	6.56	5.06	33.2	2.9	4.0	2640	3860
Average	6.51	5.11	33.3	3.2	4.5	2570	3674
3—Gravel as received including dust of fracture							
A	6.50	5.18	33.6	3.8	4.4	2550	3530
B	6.55	5.12	33.5	3.0	3.9	2720	3855
C	6.54	5.11	33.4	2.8	4.0	2495	3955
D	6.48	5.13	33.2	3.3	4.8	2575	3815
E	6.52	5.11	33.3	3.0	4.3	2800	3885
Average	6.52	5.13	33.4	3.2	4.3	2630	3788

*Concrete designed to contain 6.5 sacks of cement (5.5 portland and 1.0 natural) per cu yd, with slump of 3 ± 0.5 in., air 4.5 ± 0.5 percent, and b/b_o of 0.74. Coarse aggregate gravel graded from 1½ in.-No. 4; sand from same source used graded as received (FM 2.65).

For Condition 1, gravel was thoroughly washed and sieved to remove all minus No. 4 material; for Conditions 2 and 3, dust removed from the gravel in sieving was replaced in its original quantity to constitute 4.3 percent by weight of the total coarse aggregate.

be removed by finish screening, would be worthwhile as a measure of the value of finish screening."

I submitted calculations showing the extremely narrow range in grading of the coarse aggregate portion of a total aggregate which could result from combining the extremes of accepted grading specifications. The results of those calculations are shown in Table H. Even with these absolute extremes (which could not occur except on paper), the variations in fines in the aggregate are very small. The range in fineness modulus of total aggregate is only about 5.5 to 5.8 for the 1½ in. combination and about 6.0 to 6.3 for the 2-in. Furthermore, much of this difference exists because of variations on sieve sizes which are not controlled by "undersize" limitations.

Limited data have been secured on changes in grading of typical coarse aggregates during stockpiling and handling. These are given in Table I. Positive differences in percentages passing, and negative differences in fineness moduli, in the third lines of each portion of the table, indicate degradation of the aggregate during handling. The results show increases in coarseness as often as decreases, suggesting that the changes are largely, if not entirely, due to random sampling and testing variations. In any event, neither the changes in fineness nor in amount of fines are large enough to have a measurable effect on concrete properties. To cause operations such as these to be required to adopt finish screening because of implications in an ACI recommended practice would be completely unjustified.

There are some much less serious matters in the recommended practice that pertain to grading. The committee suggests that the ratio of maximum

TABLE H—EFFECT OF UNDERSIZE ON GRADING OF CONCRETE AGGREGATE

A. Combination of three sizes to produce combined aggregate graded to 1½ in. maximum

Sieve size	¾-1½ in., ASTM Size 4, 33 percent of total	No. 4-¾ in., ASTM Size 67, 32 percent of total	Fine aggregate, ASTM C 33, 35 percent of total	Range for extremes of combination
	Percent passing			
1½ in.	90–100	100	—	97–100
¾ in.	0–15	90–100	—	64–72
⅜ in.	0–5	20–55	—	41–54
No. 4	(0–5)*	0–10	100†	35–40
No. 8	(0–5)*	0–5	90†	32–35

B. Combination of four sizes to produce combined aggregate graded to 2 in. maximum

Sieve size	1-2 in., ASTM Size 3, 35 percent of total	½-1 in., ASTM Size 5, 21 percent of total	No. 4-½ in., ASTM Size 7, 14 percent of total	Fine aggregate, ASTM C 33, 30 percent of total	Range for extremes of combination
	Percent passing				
2½ in.	100	—	—	—	100
2 in.	90–100	—	—	—	97–100
1½ in.	35–70	100	—	—	77–90
1 in.	0–15	90–100	—	—	63–70
¾ in.	(0–15)*	20–55	100	—	48–61
½ in.	0–5	0–10	90–100	—	43–48
⅜ in.	(0–5)*	0–5	40–70	—	36–43
No. 4	(0–5)*	(0–5)*	0–15	100†	30–35
No. 8	(0–5)*	(0–5)*	0–5	90†	27–30

*All of smallest sizes assumed to be minus No. 8 to produce maximum of undersize in combined aggregate.
†Fine aggregate grading assumed constant since it would be unaffected by controls on coarse aggregate undersize.

to minimum size of coarse aggregate (I assume these are the nominal sizes) should not be more than three for sizes smaller than 1 in. and not more than two for larger sizes. I would be in general agreement with this statement if it were not followed by recommending such nonstandard sizes as ½-1¼ in., 1¼-2½ in., etc. and insistance on not recognizing the closely graded No. 4-¾ in. standard size, because the ratio of ¾-0.185 in. is more than three.

Standard sizes of aggregates have been well established through the activities of the Division of Simplified Practice of the Department of Commerce, the American Society for Testing Materials, the American Association of State Highway Officials, various departments of the federal government, and others. Agreement on standard sizes has been reached by these diverse organizations. Specifications of most state highway departments and other specification writing bodies, the American Railway Engineering Association for example, are in line with these agreed-on standards.

It would seem that the committee might at least have written its recommendations around these recognized and widely accepted standards. In

TABLE I—COMPARISONS OF AGGREGATE GRADINGS BETWEEN AGGREGATE PLANT AND CONCRETE PLANT

Item	Percent passing								Fineness modulus
	1½ in.	1 in.	¾ in.	½ in.	⅜ in.	¼ in.	No. 4	No. 8	
Company A, No. 2 coarse aggregate, average for five pairs of tests									
At aggregate plant, A	100.0	71.6	6.8	0.7	0.4	—	0.3	—	7.92
At batcher, B	100.0	68.6	6.3	0.8	0.7	—	0.6	—	7.92
Difference, B − A	0.0	−3.0	−0.5	0.1	0.3	—	0.3	—	0.00
Company A, No. 1 coarse aggregate, average for ten pairs of tests									
At aggregate plant, A	—	100.0	89.1	32.8	7.8	—	0.3	—	7.03
At batcher, B	—	100.0	91.0	40.0	9.9	—	0.9	—	6.98
Difference, B − A	—	0.0	1.9	7.2	2.1	—	0.6	—	−0.05
Company B, 1¼-⅜ in. coarse aggregate No. 3, average for eight pairs of tests									
At aggregate plant, A	100.0	—	33.1	—	0.8	0.3	—	—	7.66
At batcher, B	100.0	—	29.0	—	0.8	0.3	—	—	7.70
Difference, B − A	0.0	—	−4.1	—	0.0	0.0	—	—	0.04
Company B, ⅜-¼ in. coarse aggregate, No. 3, average for eight pairs of tests									
At aggregate plant, A	—	—	100.0	—	42.7	2.2	—	0.3	6.55
At batcher, B	—	—	100.0	—	44.9	2.6	—	0.3	6.53
Difference, B − A	—	—	0.0	—	2.2	0.4	—	0.0	−0.02
Company B, 1¼-⅜ in. coarse aggregate No. 4, average for six pairs of tests									
At aggregate plant, A	100.0	79.9	41.8	4.7	2.3	0.6	—	—	7.55
At batcher, B	100.0	74.4	37.4	4.9	1.5	0.4	—	—	7.61
Difference, B − A	0.0	−5.5	−4.4	0.2	−0.8	−0.2	—	—	0.06
Company B, ⅜-¼ in. coarse aggregate No. 4, average for ten pairs of tests									
At aggregate plant, A	—	—	100.0	76.7	36.2	1.6	—	0.3	6.72
At batcher, B	—	—	100.0	70.2	31.6	2.0	—	0.4	6.67
Difference, B − A	—	—	0.0	−6.5	−4.6	0.4	—	0.1	−0.05

general, ACI does not deal with specifications for such material—and I recognize that a specification was not proposed, but, rather, a recommendation.

I will refer to a few other matters concerning aggregates and then close without belaboring other recommendations which I feel the urge to discuss such as the "wheelbarrow and platform scale setups" and others pertaining to the concrete plant.

The report recommends that wet aggregate should be drained, with which I agree. As an alternate, it suggests, most casually, "or mechanically filtered." That implies that filtering devices are available. Later, the report states: "Mechanical equipment for filtering unstable moisture from sand apparently is becoming available, and its use should be investigated and encouraged." Where and from what source? I know of one company which did some development work along these lines, but it is my understanding, not documented, that the project was dropped. At any rate, I know of no commercial installation.

This is only another example of the off-hand manner in which the report treats many operations without giving the reader constructive information as to the why or how of the proposed practice.

I could, of course, have saved myself much trouble and avoided irritating some of my friends on the committee by not preparing this discussion. Further, I realize that it will not be printed with the recommended practice and, therefore, its preparation appears to be all the more futile. But I felt that, in justice to the Institute I would have been remiss had I not prepared it.

An amendment requires a nine-tenths vote to cause it to be sent to letter ballot. In spite of the small chance of receiving such a majority I am giving below an amendment which I proposed to the committee by letter.

Revise Section II3 to read,

"3(a). It is essential for effective control that handling operations be such that variations in undersize material outside each designated size be held to a practical minimum. The gradation of the aggregate as it enters the concrete mixer should be uniform and within specification limits. Sieve analyses of the coarse aggregate should be made frequently to assure that grading requirements are being met. When two or more sizes of coarse aggregate are being used, changes in proportions can sometimes be made advantageously to improve the over-all grading.

"3(b). If the fineness modulus of the total coarse aggregate cannot be maintained within a range of about ± 0.20 as it enters the concrete mixer, or if specification limits on grading, particularly as to permissible undersize, cannot be met consistently, special controls should be instituted. Under such conditions finish screening at the batching plant has been found to be effective where the nature of the aggregate or the methods of handling result in excessive undersize due to segregation and breakage. One approach is to establish a screen or screens on top of the bins receiving the several sizes of aggregate. Another approach is to provide an independent screening plant through which the aggregates may be passed just before entering the bins. If rinsing is required to remove contaminants the independent plant is to be preferred to avoid introducing wash water into the bins."

Note that this stresses the importance of uniformity in gradation as emphatically as does the committee, but it does not imply that, under usual conditions, finish screening is the only way to achieve that end.

Now, Mr. President I wish first to move adoption of the amendment. If that fails I wish then to move that the report be referred back to the committee.* And, if that fails, I wish to request that this discussion be received for the record and published in the *Proceedings*.

BYRON P. WEINTZ†

The report of Committee 614 is one that I, as a producer of sand and gravel and ready-mixed concrete, have read with a great deal of interest. A report such as this can, should, and does influence the operations and economics of our entire industry through its use as a guiding pattern for specifications. Of necessity, then, it must recognize all of the factors involved in making concrete—those that are technical and those that are practical. A proper balance between the two determines the economic basis for the use of concrete as a construction material.

I feel that the committee has done an outstanding job and, with one exception, I concur in general with the report. That exception is the blanket recommendation that coarse aggregates be finish screened at all ready-mixed concrete batch plants excepting those charged directly from production plant bins. This is not, in my opinion, a practical approach. Here are some of my reasons.

First, we, as producers, are in business to make, screen, and deliver to the job, aggregates meeting grading requirements standard to the local area. We should and do expect rejection of aggregates not meeting these gradations. Our company, for instance, produces some seventeen different products, each meeting its own specification requirements. My point is that we do produce material that meets specifications, and we do put this material on the job.

Secondly, there are specific instances of job setups where the contractor at his own option realizes economic advantage by producing, hauling, or feeding his batch plant with a combination of aggregates and then making the final separation with screens on top of the plant. This, however, is not an economical operation for a commercial ready-mixed concrete operator. He is, in most areas, operating on limited ground area, and must purchase his aggregates by specification gradings in relatively small quantities. His aggregates are tested and accepted or rejected on the basis of bin samples at the batch plant.

A third point is the sizeable cost of converting existing batch plants to accommodate finish screens. Many commercial batch plants now furnishing

*The amendment was put to a vote of the convention but did not receive the necessary nine-tenths of votes cast. Subsequently, the convention voted to accept the report and refer it to letter ballot of the membership—EDITOR.

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economically priced concrete in strategic locations in the cities are operating under zoning variance and "grandfather" clauses prohibiting plant addition or expansion with additional restrictive noise and dust control limitations. The possible loss of these plants would certainly affect the price of concrete.

The cost of installing finish screens on the majority of the commercial plants would be magnified due to limited plant area available to accommodate raised and lengthened feed conveyors, most of which are already set at maximum gradients. The cost of the screen itself would be, in most cases, only a small part of the total cost of installation.

A fourth consideration is the exact significance of the effect of undersize, which is debatable. To date, there seems to be insufficient data available to justify a blanket recommendation for finish screening. I, for one, would like to see further tests undertaken on this subject.

To sum up, I believe that the aggregates we produce should be subject to inspection and specification requirements f.o.b. the job and/or in the ready-mixed concrete batch plant bins. It is solely our responsibility to take whatever measures are necessary to see that this is accomplished.

By D. K. WOODIN*

Anyone experienced with the inspection and control of concrete is aware of what happens when concrete is unworkable, especially from a contractor's standpoint. It is a rare occasion in my area when screen analyses of aggregate being used for concrete meet specification requirements, but a peaceful and productive day for the contractor when they do.

To proportion a mix to accommodate a wide range in aggregate grading, results not only in a more expensive concrete, but concrete of lower quality. My files contain screen analyses of aggregates from three major aggregate producing companies in the Bay Area, for as far back as 1951, and the variations in grading make it necessary for our concrete mix to contain 0.10 bbl more cement than would be necessary for concrete of equal strength containing the specified graded aggregate.

The time has arrived when a specification requiring finish screening over the batching bins is economically feasible. I have discussed this possibility with one of our aggregate suppliers, and he agreed that finish screening would result in a better graded material, but that it would also mean an increase in cost to the consumer. This can be justified by a saving in cement and superior uniformity of concrete quality. "The economic implication of screening on every established ready-mixed concrete plant" as pointed out by Mr. Walker, does not concern the supplier mentioned.

If such a change as finish screening at the batching plant is adopted, the importance of undersize in the material as produced will have less significance. What good is an elaborate screening plant, if during the process of getting

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the material from the screening plant to the batching bunkers, the grading is destroyed? Why is it not desirable (since it has on numerous occasions been demonstrated to be practical and beneficial) to finish screen at the batching plant. Move the screening where it will do the most good, and let us adopt some progressive thinking along these lines, and abandon the "good enough" attitude.

COMMITTEE CLOSURE

Committee 614 is gratified that the discussers, representing both producer and consumer interests, have found the proposed recommendations so generally acceptable. It is quite apparent, however, that one group is not in sympathy with proposals concerning sizing and finish screening, at the batching plant of coarse aggregate, to assure its uniformity as batched.

Perhaps it is worth noting that this opposition comes from representatives of concerns engaged in producing and selling aggregates and concrete. Essentially, they say the need for these recommendations rarely exists and where it does, the causes are readily eliminated without closer sizing and finish screening.

On the other hand, it appears these particular recommendations are especially endorsed by engineers representing various agencies that are consumers and users of aggregates and concrete. These men are responsible for the uniformity and quality of these materials as furnished. They, like many others, have frequently encountered situations, under the standard specifications extolled by the producers, in which little or nothing could be done to correct seriously varying conditions in coarse aggregate as batched. They recognize that such conditions could readily have been largely corrected had they been working under specifications which included the recommended sizing and finish screening at the batching plant. From experience they also indicate an awareness that these requirements, in the over-all operation, come at a modest price, considering the improvement they make possible in concrete uniformity, quality, strength, durability, and economy. For these expressions of endorsement from experience, the committee is certainly grateful.

Since the committee unanimously endorsed the viewpoint that these questioned recommendations greatly increased assurance of uniform concrete, and they were also unanimously approved by the Technical Activities Committee and the Standards Committee, this closure will be devoted primarily to amplifying the general desirability for these requirements which the industry has so vigorously opposed.

To begin with, reference is made to pictures mentioned of stockpiles, which are no different from many, containing layers and concentrations of fine material in far excess of permissible amounts in even the broad and tolerant specifications fostered by the industry. The stockpile conditions pictured in Fig. D, E, F, and G are not unique. This breakage, concentration, and contamination of large portions with excess fines will occur to

Fig. D—Lenses and concentrations of dirt and undersize contaminate this stockpile. Finish screening will remove this and correctly classify usable material as placed in bins of batching plant

some degree in most stockpiling. It becomes worse as the material is composed of softer, or many crushed, materials.

Regardless of casual limits in popular specifications or of the range and amount of undersize in stockpiles, *if* it were possible for the same amounts of each size in the undersize to reach *each batch* as measured into the mixers, mixes could be adjusted accordingly and uniform concrete could be obtained. But obviously, this is impossible. The amounts of these fines vary widely and inevitably. When it was attempted to use material from the stockpile in Fig. D significant undersize ranged mostly up to 15 percent and was as high as 22 percent. Similar results were noted in other stockpiled aggregate.

The question has been raised of using test results to prove the necessity of doing something about varying undersize. First, such tests would postpone action such as has been taken. But mainly they would serve no purpose because the condition is a variable of considerable extremes in time and amount. When conditions are as unfavorable as in Fig. D-G, engineers responsible for getting concrete strength and uniformity do not need test results to tell them the job is in trouble.

However, one group of tests can be cited to show the effect of excess fines in the aggregates. During construction of Melones Dam in California in 1926, aggregate storage conditions were such that at times excessive amounts of fines entered the mix. The average strength of 64 test cylinders from such concrete was 26 percent less than the strength of other concrete which itself was often far from ideal.

A leader of the gravel industry expressed the fear that the recommended corrective finish screening of aggregate at the batching plant would destroy the value of the elaborate screening, handling, and blending equipment now operating in many modern aggregate production plants. This is doubtful, inasmuch as considerable initial preparation of aggregates is necessary that cannot be done with finish screens at batching plants, even when rinsing is necessary. But, one committee member comes to the real point of the matter when he asks, what good is an elaborate screening plant if, during the process of getting the material from the screening plant to the batching bins, the grading is destroyed?

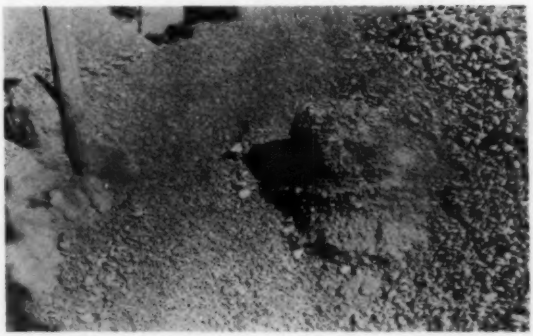

Fig. E—Mingled sizes in overlapped stockpiles as well as excess of undersize can be corrected by finish screening at the batch plant

Full recognition is given to the excellent and earnest efforts of the aggregate industry to produce well sized, uniformly graded aggregate free of excessive amounts of undersize. Unfortunately, the material seldom remains in this condition as it is handled, stockpiled, transported, and rehandled.

The principle industry discussion, including proposed revisions of Section 113(a) and 3(b), was presented to the committee meeting in Chicago, February, 1958. The proposed revisions were rejected unanimously.

The proposed revision of 3(a) sounds on first reading like an excellent statement of what should be done to get good results. Its hollow falsity becomes evident on more careful reading of each sentence. Then the question arises as to how this is to be accomplished effectively, and it is realized by anyone of experience with such matters that it is seldom practical or possible to do so to a sufficiently effective degree. The proposed revision erroneously implies that such efforts can be sufficient and effective in compensating for aggregate undersize, breakage, and variable segregation, when actually, they seldom can be.

Fig. F—Stockpile containing great and variable excesses of dirt and undersize as a result of improper stockpiling of a relatively weak aggregate. Significant undersize ranged up to 22 percent in material as batched, and made it impossible to produce uniform concrete

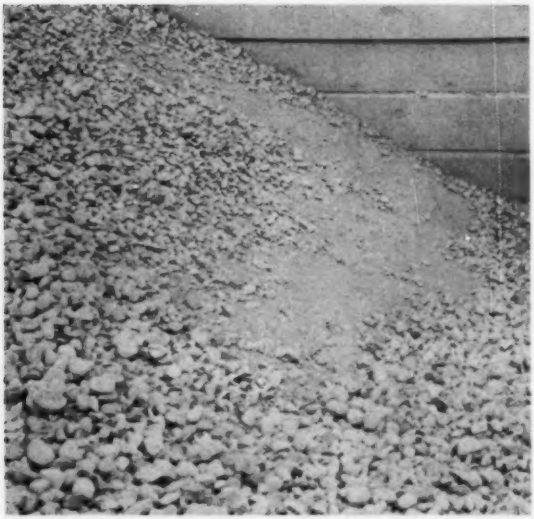

Fig. G—Excessive amount of undersize in gravel in a batcher supply bin. Finish screening removes such undersize

The proposed revision of Section II3(b), surprisingly, agreed that finish screening was a good way to correct excessive undersize, and indirectly recommend it when specification limits on undersize cannot be met consistently. Here again this seems like a faultless proposal until its uselessness is recognized in the fact that finish screening facilities cannot be installed on a moment's notice when the need arises. Few customers or projects are in a position to require installation of finish screening when its need becomes especially evident after construction has started. The proposed revision failed to provide that such plants, as may need it, be ready to finish screen when the need arises. Essentially, it says finish screening is a solution for the problem of excessive undersize but it doesn't say that by the time the need is apparent, after the job starts, it will usually be too late and infeasible to do anything about it. On the other hand, the committee's recommendation says it is usually a good idea to be prepared to finish screen, because it is often needed and well worthwhile.

Finish screening facilities are also of value to the producer and some have recognized a number of specific advantages. Many contractors have installed finish screens on their batching plants of their own volition, evidently feeling they were adequately repaid by these advantages. The ready-mixed concrete plant shown in Fig. H was equipped with a horizontally operating finish screen over the bins to qualify under the specifications to deliver about 12,000 cu yd of concrete. The producer chose to install it on the plant rather than on the ground nearby and has found this arrangement increased its benefits to him. After it had been in operation about 6 months, during which time both project and commercial concrete had been supplied, the operator

had recognized the following advantages from his standpoint. These are significant and of timely interest.

1. Never have to waste contents of gravel bins because of mistaken delivery of sand into them.
2. Never have to waste contents of gravel bins because the wrong rock size is mistakenly placed in them. Removes concern as to position of gates and chutes.
3. Permits most efficient use of storage area without expense to prevent overlap of different aggregates.
4. Permits complete use of material without waste to avoid dirt which may be picked up at bottom of piles.
5. Permits more efficient and closer operation of front end loaders carrying material from stockpiles to hopper of bucket elevator. Trips are shorter.
6. Automatically places aggregate in the center of the bin in a manner that results in a minimum of segregation, as compared with usual delivery from ordinary chutes direct from bucket elevator.
7. Reduces extremes in amounts of surface moisture to a minor tolerable variation that results in materially easier and better slump control and uniformity.
8. Blends high and low surface moisture content on small and large aggregate to a relatively uniform moisture content that materially improves slump control.

These advantages alone should be worth the few cents per cubic yard that finish screening would cost in a productive plant, not to mention savings in cement when working to a strength specification. One committee member, a producer of ready-mixed concrete, estimated that finish screening would cost about 3 cents per cu yd when averaged out through several year's production. On smaller plants it could be a few cents more but certainly would not cost $1 per cu yd as one industry representative intimated during discussions.

Cost to protect batching equipment from objectionable vibration in some plants is sometimes offered as an objection to finish screening. This condition can be corrected inexpensively by independently supporting the weighing equipment.

It is interesting to note in Table E of one producer-representative's discussion that in 50 percent of his plants, finish screening would have eliminated excessive undersize. It can be arranged to keep sand out of the gravel bins and to keep aggregate smaller than 1½ in. free of significant undersize in excess of 2 percent.

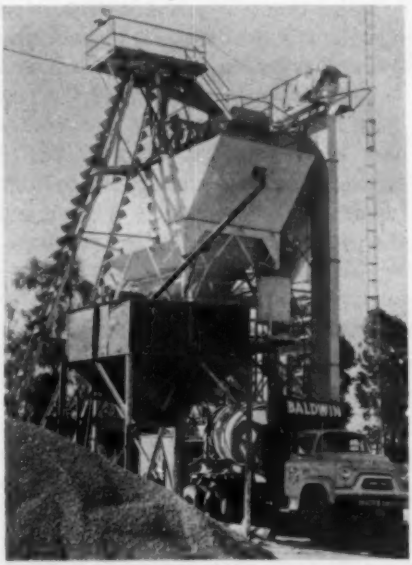

Fig. H—Horizontally operating finish screen mounted on commercial ready-mixed concrete batching plant. Undersize passes through pipe to waste bin at lower left.

When aggregate is not sufficiently clean, the finish screens provide a ready means for rinsing or rewashing. Contrary to initial fears that slump and water control would be considerably and adversely affected, such washing at at least four plants known to the writer did not present serious difficulty when operations got underway. Plant men learned adjustments required in the first few batches. Rinsing on the finish screen would allay any question of dust nuisance in zoned areas and to some extent quiet the operation.

As indicated above, one operator found that finish screens permitted more efficient use of his storage space. Elevators require less space than belts. Capacity can be increased and confusion eliminated by using a separate elevator for sand.

In this closure no attempt has been made to refute various statements in discussions by the producer group, which the committee and discriminating consumers and material engineers would question on grounds of either actual fact or negligible importance. It seemed necessary, however, to put the proposed revisions of Section 113(a) and 3(b) in their proper light. It is believed the facts are amply evident to any who are acquainted with these problems. Certainly, the expressions of the consumer group, in their unanimity and experience, can be regarded as expert testimony. Each is qualified in these matters and is in a position to say whether the recommended procedures are worthwhile.

It has been our primary purpose to show that finish screening at the batching plant is not only usually worthwhile, but that the committee is offering sound advice when it recommends that it be provided *in advance of need*, so that it will be there when the need arises. Usually it is not long until it does, if for nothing more than to keep sand out of the rock bins and to get each coarse aggregate into the right bin.

The committee also wishes to thank those who approved adoption of the report at the Los Angeles meeting and who voted on the letter ballot to adopt the proposed recommended practice. In so doing affirmation was also given to the principle that it is the responsibility of an ACI working committee to set forth and describe in its standards, procedures that are among the better ways of insuring concrete work of good quality, and not merely recite common practice.

Discussion of a report by International Council for Building Research (Eduardo Torroja):

Load Factors*

By THEODORE F. COLLIER, MAX HERZOG, EMILIO ROSENBLUETH, and EDUARDO TORROJA

By THEODORE F. COLLIER†

The choice of the title "Load Factors" for Professor Torroja's report is too restrictive. The discussion clearly includes consideration of losses of strength along with the variables in magnitude and effect of loading. The whole field of safety provisions is presented and, as Professor Torroja states, has for its goal to fix a satisfactorily low upper limit to the probability of collapse commensurate with the consequences of failure.

The introduction of ultimate strength and limit design procedures emphasized the need for properly designed safety provisions. "Factor of safety," as used with working stress design, provides a "receptacle" for unknowns and variables. "Load factor" was introduced with ultimate strength design for a similar purpose. Both perform their function but do so at considerable cost of uncertainty and inconsistency. Neither device will provide a probability (or improbability) of failure which is consistent and predictable.

What is needed is a relationship or ratio between a predicted strength as high as knowledge and quality permits and load effects having a low expectancy of being exceeded during the service life of the structure.

DISCUSSION

The report describes a direct and analytical approach to the subject of safety provisions in structures. By breaking the design procedure into its steps or phases and listing the sources of error in the essential data used in design, the controls needed to minimize and control the probability of failure become evident.

Fundamentally, the design process has two parts—the determination of the forces to be resisted and proportioning a member having at least that capacity. Those errors and variables which increase the forces as well as those which decrease the capacity may cause failure. The potential sources of errors and variations would fall under the two design steps thus:

I. Sources affecting the predicted strength of a member
   - A. Strength and behavior of the materials
   - B. Physical dimensions
   - C. Quality of workmanship and supervision
   - D. Validity of stress distribution assumptions

*ACI Journal, V. 30, No. 5, Nov. 1958 (*Proceedings* V. 55), p. 567. Disc. 55-36 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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II. Sources of error and uncertainty in use and service
   A. Loads and overloads
   B. Accuracy of structural analysis
   C. Predictability of structural behavior
   D. Consequences of failure or loss of serviceability

Since there is no relationship between the phenomena which may reduce the strength and those producing overloading, the simplest and most "visible" procedure is to apply the corrective factors and controls separately at the sources of the deviations.

This permits the designer to work with and "see" the actual value of both the predicted resistance and the anticipated load effects.

Strength losses

The sources of strength loss listed in Group I can be evaluated by scrutinizing the form and terms of the equations used to predict strength.

The form of the equations is based on certain assumptions of stress distribution and material behavior at failure or ultimate strength. The accuracy of these predictions *when the properties of the materials are known* has been adequately confirmed by recent tests and by reappraisal of former tests. The variables are in the terms of the equations, not in their form.

The terms in the equations fall into two groups, those defining the behavior of materials and those of dimension. Values of any which differ from those assumed are possible sources of understrength.

There is but one source for dimensional errors—construction. None are significant except the position of reinforcement in some members and locations.

Discrepancies in assumed values of material strength are those typical of manufactured products. For concrete the variables may be in manufacture testing, or placing. To select with reasonable assurance some minimum value of expected strength requires control of quality and knowledge of the range of variation within that quality.

The mathematics of probabilities has been applied to test data from laboratory programs and construction projects and provides a useful means of measuring strength variations for a wide range of quality control. It is feasible to classify the degree of control from test data analysis. Similarly, the expected deviation from a specified or mean value for a specific quality can be predicted. This is a useful tool for providing the needed relationships between expected strength and controlled strength and between expected strength and quality control. The committee report describes this application and has chosen many of its factors from these probability relationships.

To avoid the complexity of factors and adjustments to factors as suggested in the report, the measures for including the strength relationships in a code could consist of a table of "permissible" stresses for one or more grades or qualities of concrete with the specifications and definitions for those grades and their controls.

The dimensional adjustments would be included as specific instructions where applicable.

Load effects and analysis

The determination of the design criteria which measures some acceptably low probability of occurrence during the life of a structure is subject to many diverse variables. A study of these potential sources of error indicates that the laws of probability have little application.

Within the Group II list are two distinct divisions—those variables inherent to the loads and use, and those arising from the structural analysis and computations of the load effects.

Where the computation of moments and shears are by exact methods, the errors are well within the accuracy arising from the assumptions required by the method. When empirical coefficients are permitted, they are, or can be made, conservative within the range of application permitted.

The prediction of load variation *above* some mean or normal value poses the difficult problem. In the first place, there is no set of *mean* live loads. There are legal and recommended unit live loads which differ code by code. To apply multiplier factors to these simply exaggerates the existing discrepancies. Any set of overload factors must be based on some uniform schedule of loads. This standard schedule should be selected for some predetermined probability of occurrence. If the chosen unit values are sufficiently high, correspondingly low factors will suffice to provide for the increment of probable overload.

Live loads

The fundamental principle governing the report and this discussion requires that unrelated variables be treated separately. This would require the separation of live and dead loads as a general rule. Just as the sources of live loads differ so does their range of variation. Some are similar to dead loads in predictability. It appears then that some classification other than the present general one is needed if all live load effects are to approach a limiting value uniformly. The alternate is, as mentioned previously, the selection of the unit live loads such that the expected variation would be similar to that for dead loads. A common load factor for both could then be applied.

Under existing conditions in this country any system of live load factors should be referenced to a specific schedule of live loads and separate load factors for dead load provided.

Special hazards

The provisions for adjusting safety to the consequences of failure are best made in the unit live loads or factors for the occupancy creating the hazard. Columns are critical for many reasons and their predicted strength should be arbitrarily made conservative, either by their strength equations or permissible design stresses.

General comments

When separate adjustments are made for several variables, consideration must be given to the probability of their simultaneous occurrence. Factors and controls must be designed not as isolated values but as contributing parts of the whole system.

It should be noted that the system of safety provisions proposed will result in varying over-all safety ratios. Furthermore, these ratios will be "invisible" and "automatic" since they will not appear as values or factors, but will exist as a result of separate application of strength reduction and load adjustment. This is desirable since the resulting probabilities of failure will be less susceptible to random fluctuation caused by design and construction and will more nearly measure the protection of life and property.

By MAX HERZOG*

The Committee on Load Factors of the International Council of Building Research is to be congratulated on its effort to put forward a logical approach to the basis of the numerical values of our factors of safety. But in my opinion, the recommendations made in the report of this committee still lack the full logical consequence.

A structural element will fail if the actual strength of the construction material will be so low under the value assumed in the design calculation, and/or the actual stress will be so high above the value assumed in the design calculation, that actual strength becomes equal to actual stress. The occurrence of such a state depends on the *probability of deviations* from the design values for both strength and stress. Under the assumption that strength and stress have normal Gaussian distributions, as proved by experience (by appropriate transformation any distribution can be transformed into a normal Gaussian one), the probability of a certain deviation can be calculated if both the mean value and the standard deviation are known. So, the only thing still to be done, is to fix the *allowable probability of a failure*. Prot[1] has shown that this requires close cooperation with the insurance institutes concerned. When damage in case of failure amounts to the construction cost of the structure that failed, he proposed, on the basis of his investigation, an allowable probability of failure of 1 in 1000; for damage amounting to 10 or 100 times the construction cost, he recommended allowable probabilities of failure of 1 in 10,000 or 100,000, respectively.

Thus, it theoretically seems possible to evaluate *factors of safety* on the basis of a probabilistic analysis of all factors influencing both stress and strength.

1. Factors influencing stress:
   (a) Loads (dead, live, and metecrological load; earth and water pressure) and foundation reactions (soil pressure, bearing capacity of piles)
   (b) Design calculation (assumption of the static system, most unfavorable loading, stress computation)

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2. Factors influencing strength:
   (a) Strengths of construction materials (concrete, steel, variation of strength with the passage of time)
   (b) Workmanship (strictness of control)

To evaluate the factor of safety from the data available, Russian engineers introduced *coefficients of uniformity*.[2] The coefficient of uniformity is, for a given probability, a measure of the possible deviation from the mean value. An example will illustrate what is meant.

Mean value, $M = 1.00$
Standard deviation, $\sigma = 0.15$
Distribution is normal Gaussian

Allowable probability of being exceeded, $p = \frac{1}{1000}$

Auxiliary factor $\left(\text{for } p = \frac{1}{1000}\right)$, $F = 3.09$

Coefficient of uniformity, $K = M(1 \pm F\sigma)$.
The plus sign applies to possible increases of stresses (Index 1, p. 1390), and the minus sign to possible decreases of strengths (Index 2, above). Thus, we obtain

$$K_1 = 1.00\,(1 + 3.09 \times 0.15) = 1.46$$
$$K_2 = 1.00\,(1 - 3.09 \times 0.15) = 0.54$$

The greatest numerical value of the factor of safety S is obtained when the chosen allowable probability of failure p is divided among the four main factors influencing stress and strength in such a manner that the product of the uniformity coefficients of stresses, K_{1a} (loads and foundation reactions) and K_{1b} (design calculation), divided by the product of the uniformity coefficients of strengths, K_{2a} (strengths of construction materials) and K_{2b} (workmanship):

$$S = \frac{K_{1a}\,K_{1b}}{K_{2a}\,K_{2b}}$$

becomes a maximum. Since S, K_{1a}, K_{1b}, K_{2a}, and K_{2b} are functions of p, there must hold:

$$\frac{dS}{dp} = 0, \quad \text{and} \quad \frac{d^2S}{dp^2} < 0$$

Summarizing, the writer wishes to point out that he prefers his own method of evaluating the factor of safety to that proposed by the committee, because the committee proposal at first implicitly assumes a standard probability of deviation from the mean value of 1 in 6.3 for both load and strength by putting $F = 1$, and later on introduces corrections of over-all nature to better take care of reality, but *without* mentioning the probability of failure, underlying the recommended numerical values. The writer in his method explicitly defines the risk in form of the allowable probability of a failure at first. Then, the factor of safety is calculated with the help of the coeffi-

cients of uniformity, thus making evident how the factor of safety really depends on the possible deviations from the mean values—as introduced in the design calculations—of all factors of influence.

Finally, it should be mentioned that the adoption of factors of safety on the basis of a probabilistic analysis requires specifications reflecting this new viewpoint. Not only the mean value of strength alone should be specified, but the mean value and the allowable deviation, or the minimum value for a given probability of not being reached. The latter formulation leaves full freedom to the contractor in fulfilling specified strength requirements. To control the use made by the contractor of this freedom, reductions of the price of construction and penalties should be fixed in advance as adequate recompense for reduced usability of the structure, in case that the specified strength is not reached.

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BY EMILIO ROSENBLUETH*

The distribution of a combination of random variables can be obtained through application of standard probability theory[3,4] or by the method of random products. An approximate method,[5] consists in taking the factor of safety for a combination of variables as given by the expression

$$C = 1 + \sqrt{\sum_i (C_i - 1)^2} \qquad (1)$$

where C_i is the individual factor of safety for the ith variable, that is, C_i would be required if the ith were the only random variable in order to give a prescribed probability of failure, and C is the safety factor required when all the variables are random. These factors of safety are taken with respect to expected values; to obtain the safety factor relative to nominal values one must divide by the product of ratios of expected to nominal values of the variables.

Application of this method to the problem of the safety factor of timber structures, studied by Wood by the method of random products,[6] gives an error of 3 percent on the safe side.

With inspiration in Wood's paper, the following indeterminacies will be assumed for design of reinforced concrete. All distributions will be taken as normal. Corresponding values of parameters appear in Table A. They are based on judgment and limited data and the writer trusts that the com-

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mittee will greatly improve them. Distributions assumed are admittedly incorrect but presumably satisfactory in the range of greatest interest.

Strength as measured in control specimens

The distribution for the steel yield point is frankly skewed and can probably be represented by Weibull's distribution.[7] For the present application it will be taken such that about 5 percent of samples fall below specification. For concrete the nominal strength will be assumed equal to the mean; the corresponding standard deviation could perhaps be expected to fall between 0.07 and 0.14 of the mean, depending on the type of control.[8] Indications are that the distribution may approach the extreme I-type for strengths much lower than the mean.[4]

In-situ strength in terms of strength of control specimens

For concrete, size and stress gradient effects combined with the effects of adequate placing, curing, and vibration will be assumed to give a distribution with mean equal to the nominal value and coefficient of variation 0.05. At first glance a 15 percent reduction in the mean would seem justified, but it is assumed that such a reduction is systematically made in the formulas used to compute ultimate strength. For steel some reduction is expected from such practices as bending of bars, splicing, and welding. Admittedly a normal distribution can only be expected for reductions, not increases in strength. It should be noticed that the standard deviation of in-situ strength for steel is a decreasing function of the number of bars and the degree of statical indeterminacy. The mean should be reduced and the spread of reductions due to placing should be increased beyond the values assumed when expected construction practice deserves to be classified as average.

Use of standard sizes

Rounding off to whole inches in dimensions of concrete members is done in such a manner as not to overstress excessively the concrete, hence the parameters assumed in Table A. Since bar areas differ by 30-60 percent in successive standard diameters of American practice, the spread in the case of columns with either an even number of bars or a multiple of four bars is large. To a lesser extent the same is true of sections of maximum moment in beams, but the spread is quite small in sections close to the points of cutoff of longitudinal bars. Again for steel it is not unlikely that a Weibull distribution would prove more appropriate, and the parameters certainly should be varied with the member size and type and with the bar diameter.

Geometry of actual structure versus drawings

In this connection, usual practice introduces both systematic and random differences between specified dimensions or position of reinforcement and actual values. Both types of variation are essentially independent of the size of the member.[4] Hence it is not reasonable to assign to these variations a distribution in terms of a percentage of nominal dimensions unless the nominal dimensions are also explicitly assumed. Generally speak-

ing, total depths of members may be expected to be, on the average, 0.2 in. greater than the specified depths with normal distribution and a standard deviation of 0.4 in. Widths would be subject to lesser variation. Effective depths for negative steel, at least in slabs, are systematically smaller than what the drawings show (about 0.35 in. smaller), are normally distributed, and have a standard deviation of about 0.4 in.[4] For columns, the cross-sectional area would be expected to be systematically a little larger than nominal, with deviations independent of size. In a specific type and size of member, on this account one could find the distributions assumed in Table A.

Wear

Due to blows, abrasion, and mild corrosion, resistance may be lowered. Ordinarily smaller members are apt to suffer relatively more from these causes than larger members. Under favorable conditions there should be no reduction on this account.

Loads

As stated by the committee, the probability distribution of the loads depends greatly on the type of loading. For a member subject to relatively small loads a nominal load equal to the mean and a coefficient of variation of 0.10 seem reasonable. Conditions would in general be more favorable for members subject to greater expected loads.

Duration and repetition of loading

It will be assumed that steel is unaffected by these factors but that the average strength of concrete, relative to that manifest in static loading, is reduced 10 percent with a standard deviation equal to half the mean reduction.

Stress analysis

Variations due to this concept will be included in factors with means and standard deviations as listed in Table A. When stability is governed by shear, the factors must be assigned a much greater spread.

The following conditions were assumed in constructing Table A. (Except for subscripts and prime signs nomenclature is that of the committee.)

- A_o = All conditions met. No weathering or wear anticipated. Coefficient of variation in cylinder strengths is 0.07.
- A_o' = Coefficient of variation is 0.14.
- B_o = Same as A_o except average accuracy of computations. Parameters adjusted to give 15 percent increase in load factor over that for A_o without change in probability of failure.
- C_o = Same as A_o but with average workmanship. Parameters adjusted to give the same load factor as for condition B_o.
- D_o = All parameters as for condition A_o except in accuracy of computations (as for B_o) and workmanship (as in C_o).
- D_1 = Same as D_o but mild weathering and wear expected.
- D_1' = Parameters as under condition D_1 except the coefficient of variation in cylinder strengths is 0.14.

TABLE A—FACTORS AFFECTING SAFETY IN REINFORCED CONCRETE

Condition	Factor	Concrete		Steel	
		Expected / Nominal	Coefficient of variation	Expected / Nominal	Coefficient of variation
A_o	1	1.00	0.07	1.05	0.03
	2	1.00	0.05	0.98	0.02
	3	1.03	0.02	1.05	0.03
	4	1.03	0.05	0.97	0.05
	5	1.00	0.00	1.00	0.00
	6	1.00	0.10	1.00	0.10
	7	0.90	0.05	1.00	0.00
	8	1.02	0.04	1.01	0.03
A_o'	1	1.00	0.14	1.05	0.03
B_o	8	1.02	0.1145	1.01	0.1104
C_o	2	0.95	0.1004	0.95	0.0995
D_o	2	1.02	0.1145	0.95	0.0995
	8	0.95	0.1004	1.01	0.1104
D_1	2	0.95	0.1145	0.95	0.0995
	5	0.95	0.03	0.95	0.03
	8	1.02	0.1004	1.01	0.1104
D_1'	1	1.00	0.14	1.05	0.03
	2	0.95	0.1145	0.95	0.0995
	5	0.95	0.03	0.95	0.03
	8	1.02	0.1004	1.01	0.1104

Parameters for factors not tabulated are equal to those for condition A_o. Expected/Nominal signifies ratio of expected to nominal value. Coefficients of variation are taken as ratios of standard deviation to expectation.

Two other conditions will be considered: EA_o and ED_1', for which the parameters are as for A_o and D_1', respectively, but the load factor is raised 15 percent to correspond to potentially serious consequences of failure.

Application of Eq. (1) permits computing the load factor relative to the product of the means. This is then divided by the product of ratios expected nominal to obtain the load factor relative to nominal values. Results are shown in Table B. For conditions A_o through D_1' permissible probabilities of failure were chosen as 10^{-4} and 5×10^{-4} for concrete and steel, reflecting the different types of failure of the two materials in flexure of reinforced concrete elements. For conditions EA_o and ED_1' the probabilities of failure were obtained by trial and error to give the desired load factors.

Analogous computations were run with other reasonable sets of parameters and with much higher probabilities of failure. Substantially the same conclusions were derived as from study of results listed in Table B.

TABLE B—LOAD FACTORS AND PROBABILITIES OF FAILURE

Condition	Concrete		Steel	
	Load factor	Probability of failure*	Load factor	Probability of failure*
A_o	1.866	1	1.472	5
A_o'	2.375	1	1.472	5
B_o	2.146	1	1.693	5
C_o	2.146	1	1.693	5
D_o	2.404	1	1.878	5
D_1	2.535	1	1.981	5
D_1'	2.959	1	1.981	5
EA_o	2.146	0.035	1.693	0.186
ED_1'	3.403	0.158	2.228	0.720

*In units of 10^{-4}.

Comparison of condition A_o' with A_o and of D_1' with D_1 permits judging the validity of the committee's definition of characteristic strength as $f_c' - F\sigma$ with $F = 1$. With respect to the mean strength of concrete cylinders, the load factor for condition A_o' is 27.3 percent greater than that for condition A_o while the one for condition D_1' exceeds that for D_1 by 16.7 percent. If F is taken equal to unity the increase in load factor in both cases, due to raising the coefficient of variation of concrete strengths from 0.07 to 0.14, is $[(1 - 0.07)/(1 - 0.14)] - 1 = 0.081$ or 8.1 percent. To give the correct variation in load factors F would be required to equal 2.52 for conditions A_o and A_o' and 1.79 for D_1 and D_1'. These values are much closer to Baker's[9] proposal ($F = 2$) and in the first case even closer to the Russian building code ($F = 3$).

Another way of arriving at the desired dependence of load factor on the standard deviation of concrete strengths may consist in the use of narrow tolerances on cylinder strengths. For example, the requirement that a certain percentage of the samples should give strengths not smaller than the nominal f_c' and none below a certain percentage of the nominal f_c' can be made equivalent to adoption of as high a (variable) value of F as desired. However no verdict can be issued on the adequacy of such specifications without a refined probabilistic treatment.

Comparison of conditions B_o and C_o with D_o permits evaluating the committee's proposal that the load factor under condition D be taken as 15 percent greater than under B or C, each of which has been taken 15 percent in excess of the one for condition A to account for average accuracy of computations or workmanship, respectively. According to that proposal, the load factor for the concrete under D_o should be $1.15 \times 2.146 = 2.468$, which exceeds 2.404 by nearly 3 percent. For steel the error is near 4 percent.

Baker proposes[9] that increases in load factors (relative to $f_c' - 2\sigma$) be taken as additive, not multiplicative. For concrete under condition A_o there is a load factor of $1.866 \times 0.86 = 1.605$. If the one for conditions B_o and C_o, $2.146 \times 0.86 = 1.846$, is taken as correct, an increment of $1.846 - 1.605 = 0.241$ is due either to average workmanship or average computational accuracy. When both concepts are average the load factor relative to $f_c' - 2\sigma$ should be $1.846 + 0.241 = 2.087$, which amounts to $2.087/0.86 = 2.427$ relative to f_c', introducing an error of 1 percent with respect to the formerly computed 2.404. For steel one finds by Baker's criterion a required load factor of 1.912 instead of the actual 1.878, or an error of 2 percent. Since Baker's and the committee's proposals are equally simple, it seems advisable to follow the additive criterion, which leads to errors half as great as the multiplicative rule, although admittedly the error is not large in either case.

A different situation arises when analyzing the probabilities of failure for cases EA_o and ED_1'. Consideration of the serious consequences of failure requires a 15 percent increase in the load factors of condition A_o. This brings the corresponding chances of failure in the concrete down to about 1/30 of

what they are under ordinary conditions. The result is acceptable in view of what would usually be understood by grave consequences of failure. But the same 15 percent increase in the load factor for condition D_1' leaves the probability of failure in concrete 4.5 times greater than for case EA_o. Results for steel are only slightly less bad. Baker's criterion of adding a constant to the load factor (relative to $f_c' - 2\sigma$) to take care of the desired reduction in probability of failure would entail even greater disparity among the probabilities for cases EA_o and ED_1'.

In order to have case ED_1' with the same probability of failure as EA_o the load factor for concrete would have to be raised to 3.810, which represents an increase of 29 percent, not 15, over 2.959. It is apparent that variations in the seriousness of potential failures should not be adjusted by such simple rules as multiplication of the load factor by a correction coefficient or addition of a constant. Unless other approximate but more accurate formulas are worked out, there is need for a table of load factors to cover cases of practical interest.

Comparison of the load factors for D_o and D_1 indicates the convenience of modifying the load factor when even mild wear or weather effects are anticipated.

From the discussion of the variables that have a bearing on the choice of load factors, it is clear that vulnerability is a decreasing function of section dimensions. This is true of standard sizes, defects in geometry, effects of blows, abrasion and corrosion, uncertainty in loads, difficulty in placing concrete, and the consequences of insufficient curing. Certainly the desirability of lower probabilities in members carrying greater loads, combined with size effects in concrete strength, does not cancel the greater vulnerability of smaller members. Hence it is justified to use load factors with decreasing functions of section dimensions. This is conveniently accomplished by assuming in design effective section smaller than the nominal sections. One could, for example, reduce in computations 0.5 in. all around the perimeter of each member, reduce effective depths of negative reinforcement 0.5 in. (except where special provisions guarantee that the nominal effective depth will not be reduced), and "peel" reinforcing bars of, say, 0.04 in. all around (only when weather conditions dictate it). Such practice is no more complicated than use of characteristic strength; analogous reductions have for years been common practice in design of penstocks exposed to corrosion. Of course, load factors relative to reduced sections and to characteristic strengths should be lower than those referred to nominal values. The end result would be a set of probabilities of failure with not much more variation than is justified by variations in the consequences of failure.

It is worth noting that the use of a load factor combined with a factor of safety in concrete strength and one in steel yield point can be simplified by omiting the factor of safety in steel yield point and adjusting the other

two factors accordingly. In cases of nonlinear dependence of ultimate load on strength, the same simplification should be possible with minor adjustments.

Required load factors are sensitive to type of member. In an axially loaded column, a given change in the yield point of longitudinal steel produces a small percentage of variation in load carrying capacity. In a beam there is near proportionality between yield point of a few bars and ultimate load. Again in slabs the situation is much more favorable due to a combination of factors.

Resulting differences in required load factors can be taken in consideration in building codes by specifying correction coefficients which depend on the type of structural member, as is done in the Russian code. More simply, design formulas and coefficients may be adjusted to take care of the variations, as is done for flat slabs in the ACI Code. It seems indispensable to treat this point when specifying load factors.

The committee implicitly recommends live load reduction in terms of the number of stories carried by a column. Yet it is well established that design unit live load should vary with tributary area rather than number of stories.[10-12] Variation with number of stories is in any case a minor refinement to be superimposed on the variation with loaded area. Objections to the tributary area criterion, based on the insufficiency of data for such specifications, apply equally to the number of stories criterion.

From the present discussion the following conclusions can be derived.

1. Although available data are incomplete, there is a wealth of information in the literature. In view of this and of the complexity of the problem, it is justified to study the question by the method of random products. The writer is confident that the Committee will greatly improve on the study presented in this discussion.

2. Effects of average workmanship and average accuracy of computations are better accounted for by the use of additive constants than multiplicative correction factors.

3. Load factors should depend on the degree of exposure to wear and weathering. They should also depend on the type of member considered. This can be taken implicitly in consideration by adjusting design formulas and design coefficients in building codes.

4. A greater weight should be assigned to deviations in concrete strength than implied in $F = 1$. It seems appropriate to take $F = 2$ or greater. Narrow tolerances in minimum cylinder strength may also lead to satisfactory results.

5. A more refined treatment of the severity of potential failure is justified than multiplication by 1.15 or 0.85.

6. The greater vulnerability of small sections should be taken into consideration. This is conveniently achieved by "peeling" nominal concrete sections in design, depriving them of a skin of constant thickness (such as 0.5 in. all around the perimeter), which should only depend on expected workmanship. A similar practice is advisable for reinforcing bars when these are prone to corrosion. Effective depths of negative steel should also be assumed smaller than nominal, with a constant reduction, unless special provisions are sure to be taken to fix the position of reinforcement.

7. Design unit live loads should depend on tributary area rather than on number of stories carried by a column.

8. Rather than requiring a load factor, plus a safety factor on concrete strength and one on steel yield point, the values may be adjusted so as to permit omission of the safety factor for steel yield point.

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CLOSURE BY EDUARDO TORRAJA

Mr. Herzog deplores that the committee proposal makes no reference to the probability of failure, and for that reason he prefers systems similar to the one put forward by Prot. But in actual fact, the committee proposal is largely based on the ideas of Prot. The apparent difference lies in that the abbreviated form adopted by the committee, suitable for practical application, makes it impossible to include the laborious calculation procedure of the various probabilities of failure, which has been the basis for the determination of the final coefficients.

These probabilities of failure have not been fixed in an arbitrary or conventional adoption of a numerical factor (for instance, $p = 0.001$ for normal cases, $p = 0.0001$ for cases of grave risk).

The system adopted is based on the following principles.

1. The greater the factor of safety, the greater the cost of the structure.
2. The greater the factor of safety, the smaller the probability of failure.
3. If C_γ is the cost of the structure for a given factor of safety, D the damage due to a possible accident, and p_γ the probability of failure (which, like C_γ, is a function of the value γ assigned to the factor of safety), then the over-all cost of the structure, including the insurance premium to cover against the possible damage D, will be:

$$C_T = C_\gamma + p_\gamma (D + C_\gamma)$$

4. The factor of safety has been defined as the value γ which makes the total cost C_T a minimum.

5. In the analytical investigation, whereby the function giving the probability of failure p_γ has been obtained, the hypothesis of a Gauss distribution of errors has not been assumed. The initial experimental statistical laws have been plotted by points, in accordance with available data, including local distortions. The analytical expression of these laws has been accomplished with the aid of special mathematical methods, which were worked out for this purpose (*Cf. El Coeficiente de Seguridad en las Distintas Obras*, by A. Paez, directed by E. Torroja, published by Instituto Tecnico de la Construccion, Madrid, 1950, 210 pp.).

The author believes that the method adopted to estimate the probability of failure is more accurate than the method of random products proposed by Mr. Rosenblueth. The possibility of adding additive constants, instead of correcting factors, was discussed by the committee. The second method was chosen as it offered a better representation of the theoretical results obtained by direct calculation. It must be noted that the coefficients have not been formulated in exact accord with the theory. They only constitute a simplified representation of the results obtained in the general investigation.

Mr. Rosenblueth's suggestion that the design unit live load reduction should be expressed depending on tributary area rather than on number of stories carried by a column is interesting. As a general rule, the distribution of live loads in various stories is due to independent causes. Hence the probability of a simultaneous maximum overload is small. It is likely that the same is true for a large area within the same floor level. The difficulty lies in determining this *a priori*: this does not seem likely, except in particular functional cases. Finally, the general recommendations must always be applied after evaluating their aptitude to the particular case involved.

Mr. Collier proposes to study and evaluate the factors of safety by splitting up these factors into the various sources of error which may arise.

In his view, this system, in addition to a greater accuracy in fixing the numerical value, is more flexible in its adaptation to the various particular cases.

Actually this was the method that was followed in working out the conclusions as finally approved. Starting from statistical laws, graphs were drawn illustrating the probability that a particular variable might surpass a given value. After defining the laws describing a particular phenomenon, these had to be integrated so as to determine the over-all probability law as a function of the resulting factor of safety, representing the product of the different variables included in the over-all probability law.

This probability law or function, which gives the probability of failure for a given value of the factor of safety, defines the condition of minimum cost of insured structure, which is the objective basis for the determination of the magnitude of the factor of safety as adopted by the committee.

However, the joint probability law depends not only on the mean error of each of the parameters but also on the form of the individual statistical law of probability.

As it is practically impossible to work out in each particular case the complex set of calculations which are necessary to integrate the various probability functions, it was decided to do this for the most usual cases, and from these results the practical conclusions given in the proposal were finally obtained.

To simplify as much as possible the practical method of obtaining the coefficients, various sets of facts have been grouped together under comprehensive headings, although as Mr. Collier points out, these could be analyzed or evaluated separately. If these circumstances were subdivided more closely, and if this were to be done objectively, it would involve a statistical investigation of the errors. This would require a search and compilation of data, for a particular case, demanding much time, without thereby leading to significantly greater accuracy.

Discussion of a paper by George C. Ernst

Moment and Shear Redistribution in Two-Span Reinforced Concrete Beams*

By MILÍK TICHÝ and AUTHOR

By MILÍK TICHÝ†

Regarding the results summarized by the author I should like to draw attention to the fact that the maximum theoretically possible load for a given continuous beam is determined by the plastic hinge method, which was also used for evaluating the crushing load of the test beams. The excessive ratios of maximum applied test load to the calculated crushing load seem to indicate some error in the determination of constants appearing in the calculation of crushing moments.

The discrepancy may be explained by the accidental behavior of the moment-curvature relationship, which has an important influence on the ultimate load of a continuous beam.

We have measured the curvatures in 139 sections of 12 pretensioned simply supported beams by means of a curvature gage. We have found a coefficient of variation of curvature of about 8 percent. This coefficient was constant with increasing load. Its value would perhaps be greater for ordinary reinforced concrete beams. I suppose that a statistical approach to the solution of crushing strength of continuous beams is necessary to find the source of discrepancies between tests and theoretical results.

Regarding the manner of redistribution and its invariancy with the steel ratio, I received similar theoretical results for prestressed concrete beams.

The loading cases No. 2, 3, and 5 and their results are most interesting. In my opinion they bring something new in the investigation into the reinforced concrete continuous beams.

AUTHOR'S CLOSURE

With regard to the ratios of maximum applied test load to the calculated crushing load, the large ratios for the low-steel ratios are not due to an error in the determination of the constants for calculation of the crushing moments. The crushing moments were computed as defined in References 1 and 7, which provide values based on the attainment of an ultimate compressive

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strain corresponding to the full development of the stress-strain relationship for concrete. Such moments are not the ultimate or maximum moments for steel ratios below the balanced value (see Reference 1), but were considered more desirable as reliable analysis limits for the purpose of determining the steel ratio at which the maximum applied test load occurred simultaneously with calculated ultimate compressive concrete strains at the critical sections.

It is encouraging to know that theoretical calculations for prestressed beams tend to confirm the manner of moment and shear redistribution with regard to variations in steel ratios.

The discussion by Mr. Tichý was appreciated and also helpful in clarifying certain aspects of analysis that were not made clear in the paper.

Discussion of a paper by Jack R. Benjamin and Harry A. Williams:

Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings*

By VITELMO V. BERTERO, JACQUES COHEN and A. M. OZELL, and AUTHORS

By VITELMO V. BERTERO†

By the publication of this outstanding paper the authors have done a great service to all those interested in the design of reinforced concrete shear wall structures, which is one of the most satisfactory type of structures to resist large loads produced by atomic blasts, earthquake or wind.

This writer has had occasion to become acquainted with the excellent work previously done by the authors[3] while studying the behavior of shear walls with solid panels, under dynamic loads, at the Massachusetts Institute of Technology. The results of this study were reported in a thesis[12] and in unpublished partial reports. This background enhanced the writer's interest in the authors' paper and provided the basis for a substantial portion of the discussion which follows.

Regarding the analysis of the behavior of reinforced concrete shear walls in the elastic range, the writer agrees with the authors, that the Portland Cement Association procedure offers some inconsistencies and its application yields results which are questionable.

The method of analysis suggested by the authors which more accurately accounts for distortions seems to be a more rational approach to the problem. The accuracy of the method is well illustrated by the excellent correlation between experiment and theory for Type I walls. It is regretted that, due perhaps to the necessity of condensing the paper, the authors did not illustrate its application.

The writer agrees also with the authors that the lattice analogy solution gives better results, however, he believes that the accurate prediction of the behavior of shear walls in the elastic range is of academic importance only. Of more importance is the prediction of the wall behavior involving inelastic deformation, since under design conditions it may be desirable to allow for a certain amount of cracking and in some cases even larger deformations approaching collapse.

It is possible that the lattice analogy may be used satisfactorily under static conditions to predict the behavior of walls with openings in the inelastic range. Based on the writer's experience with this method it would appear

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that to obtain fair results it would be necessary to increase the fineness of the lattice pattern to such a degree that the time involved in its solution would make it prohibitive.

It is perhaps due to the above fact that the authors preferred to use directly an empirical method, which assumes the load-deflection relationship beyond cracking can be approximated satisfactorily by two straight lines. The first straight line gives the relationship between the cracking load and the ultimate load, and the second is a horizontal line representing deflections after ultimate load. The deflection associated with ultimate collapse, however, is not defined. From the analysis of the curves given in Fig. 4, 5, and 8 it can be seen that with the exception of Specimens H-8 and HII-4, the agreement between the experimental and theoretical curves appears to be very good.

In spite of this satisfactory agreement the writer has some doubts about the generality of the empirical relationship used by the authors to determine the deflection at ultimate load, i.e., "five times the elastic deflection that would be present if the wall acted elastically up to the ultimate load."

If it is true that elastic deflection may give some indication of the deflection after wall cracking, it seems that this cannot be taken as the only factor on which to base the deflection at ultimate load. While the wall rigidity in the elastic range is virtually a function of the dimensions of the wall, size and location of openings, and of the modulus of elasticity of the concrete and is practically independent of the amount and distribution of steel in the panel and bounding frame, this is not true for the inelastic range. In this case the rigidity and, above all, the ultimate load seems to depend to a large extent on the amount and distribution of the reinforcing. This influence was pointed out by the authors in their previous paper[3] where the behavior of wall with solid panel was discussed, and is also implied in the following statement taken from their present paper: "Long diagonal well anchored corner bars significantly increase both strength and rigidity of a wall containing openings once cracking begins."

It is regrettable that the authors did not show the theoretical load-deflection curves for HR and VRR series walls, because the writer feels that a comparison of these curves with those obtained experimentally would have shown clearly the influence of the steel.

As an example the writer would like to compare Specimens VRR-4 and VRR-5. These two walls were identical with the exception that VRR-5 had two extra #3 long, crossed diagonal bars in each pier. Consequently, the first major crack occurred under the same load, and in spite of the fact that the experimental curves showed different elastic deflections, it is logical to assume the same lateral stiffness for each specimen. The average lateral stiffness of these specimens is approximately 7.5×10^{-4} in. per kip. Using this value and the method suggested by the authors, the load-deflection relationship for the cracked range can be determined. If this is done the

load-deflection straight line for VRR-4 would be in good agreement with the experimental curve, however, for VRR-5 this could not be true. In this case the computed value is considerably larger.

Similar results are obtained if the load-deflection curves for Specimen HR-5 and HR-7 are compared with that of HR-3 or HR-4.

From the analysis of the load-deflection curves for HR and VRR series walls, it can be seen that adequate reinforcing in the piers and around openings not only increases strength and rigidity, as concluded by the authors, but also increases considerably the energy absorbing capacity of the wall at failure. For example the addition of two #3 diagonal bars in VRR-5 with respect to VRR-4 increased the energy absorbing capacity approximately 1.6 times. Additional reinforcing in the piers and the presence of two #2 diagonal bars in HR-5 instead of the two #2 extra straight side bars of HR-4, meant an increase in the energy absorption capacity of more than four times.

Ductility

The energy absorption capacity of a shear wall which is an important factor in the design of blast or earthquake resistant structures depends to a large extent on the ductility ratio. This is defined[14] as the ratio of maximum deflection at the point of collapse to the yield deflection. The deflection at ultimate load as computed by the authors could actually be considered as the yield deflection. Unfortunately there is presently no suitable method available for the prediction of the deflection at collapse.

As indicated by the load-deflection curves presented by the authors, this ratio varies from 1 for Wall VRR-1 (brittle failure) to 6 or more for Wall HR-5. Of course due to uncertainties about the rate of loading and test procedure in general these values may be only crude approximations.

Based on the experimental results obtained by the writer on walls with solid panels, it appears that the ductility ratio depends to a large extent on the percentages of steel in the panel and in the columns. Increasing the amount of steel increases continuously the rigidity and strength of the wall, the ductility ratio increases only up to a certain point and then decreases. If the percentage of steel is increased sufficiently the ductility ratio can be reduced to 1, i.e., the failure of the wall is a brittle one due to crushing of the concrete in the panel at the corner closest to the point where the load is applied.

Finally the writer would like to comment on the limitations in the applicability of the data presented, and method suggested by the authors insofar as the prediction of the behavior of the complete shear wall structure is concerned.

Other variables affecting shear strength

In the design of monolitic reinforced concrete shear wall structures difficulties may arise in applying their data due to the fact the actual load-displacement relationship of the structure is a function not only of the rigidity of the shear wall itself but of many other parameters as well which can never be accurately

determined. Among these parameters are two which are related directly with the rigidity of the shear wall. These are, the sliding and rotation of the foundation and the integral action of shear wall, face walls, and floor and roof slabs. Since the data given by the authors are for walls supported on essentially a rigid foundation, their use is somewhat limited.

The writer is aware of the fact that a primary consideration in the design of building foundations subjected to large horizontal loads, is to prevent or at least to limit both sliding and rotation to such magnitudes that these motions do not affect the behavior of the structure under static loads. While the design loads are normally considered as static loads, they actually are dynamic in character. Therefore small amounts of sliding and/or rotation may affect considerably the dynamic response of the structure.

Regarding the influence of face walls and floor slabs, it is obvious that these elements acting as integral parts of the structure may impart considerable strength and stiffness to the shear wall itself. The effect of this additional strength which was investigated by the authors in the case of solid panels[3] may become important in the analysis of relatively flexible shear walls, i.e., walls with small L/H ratio and large openings as would be the case of walls represented by Specimens H-7, H-8, and H-9.

A good illustration of this influence can be obtained by comparing the load-deflection curves of Specimens HII-3 and HII-1. These two specimens were identical except the main tension and compression columns, which simulate to a certain degree the interaction of face walls, were eliminated in HII-3. From the data given in Fig. 8 it can be seen that HII-1 was twice as strong as HII-3 and approximately 1.8 times more rigid.

It should perhaps be pointed out that the difficulty for an accurate evaluation of the above mentioned influence, lies in the uncertainty as to the effectiveness of the flange action of the face walls or slabs acting integrally with the shear wall.

Effect of vertical forces

Another factor affecting the behavior of shear wall structures is the presence of vertical forces. According to the results obtained by the writer on walls with solid panels, even a small vertical load acting downward at the top of the bounding tension column, changes the mode of failure, and consequently the value of the ultimate load and the ductility ratio. This vertical load does not allow bending cracks or diagonal tension cracks to develop, thus resulting in brittle failures due to crushing of the concrete in the panel at the corner closest to the point where the load was applied.

It is hoped that none of the foregoing discussion will be interpreted as unwarranted digression, but rather that it will be deemed to have a useful relationship to the author's excellent presentation.

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By JACQUES COHEN* and A. M. OZELL†

The authors are complimented for the excellent reporting and presentation of the tests on the behavior of one-story reinforced concrete shear walls containing openings.

Similar studies involving a full size steel frame filled with unreinforced concrete block masonry have been conducted at the University of Florida. The following excerpts from a report of this study‡ are presented for the purpose of throwing additional light on the behavior of frames containing shear walls with and without openings.

The purpose of this study was to determine the behavior of steel frames filled with unreinforced masonry walls by conducting tests on a full-scale model. Effects of openings, such as windows and doors, also were considered. A total of four tests were run, i.e., bare frame, frame with solid wall, frame with wall containing a door opening, and one with a window opening.

The testing frame consisted of a 10 × 15-ft steel bent and was supported on pins which allowed the ends to rotate. Fig. A shows the test setup with the frame filled with masonry wall containing a door opening. The amount of restraint of the column ends was controlled by applying known forces with a jack through a lever system.

The frame was loaded horizontally by a hydraulic jack and the magnitude

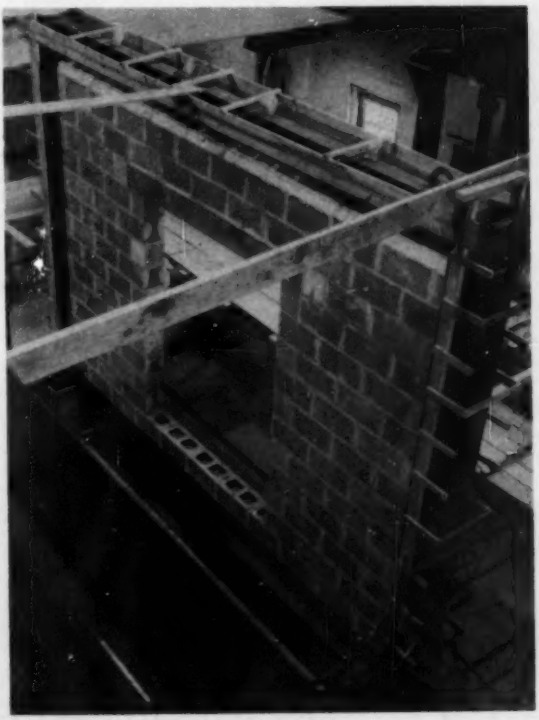

Fig. A—Frame with wall containing a door opening

*Member American Concrete Institute, Graduate Student, University of Illinois, Urbana, Ill.; former Graduate Student, University of Florida, Gainesville, Fla.
†Member American Concrete Institute, Professor of Civil Engineering, University of Florida, Gainesville, Fla.
‡Cohen, Jacques, Master's Thesis, University of Florida, 1958.

of the force was determined with the aid of a dynamometer. The side sway deflections of the frame were measured by dial gages.

Test Results

Fig. B shows the theoretical and experimental load versus horizontal deformation (H-δ) for the bare beam. Theoretical values were computed by using a numerical procedure and a value of 30 x 10^6 psi for the modulus of elasticity for steel.

Fig. C is a composite presentation of test results showing the deflected column reading for all frame conditions. The test results indicated that the frame with a solid wall was about 7.0 times more rigid than the bare frame; the frame with a wall containing a door opening was about 2.5, and the one with a window opening was about 7.0 times more rigid. All these values were based on uncracked wall conditions.

As could be expected, the behavior of each structure tested was different. However, there were some common characteristics:

(1) The column ends were considerably stiffened by the walls;

(2) The increase in rigidity of the composite structure when filled with the solid wall and with the wall containing a window opening showed a range

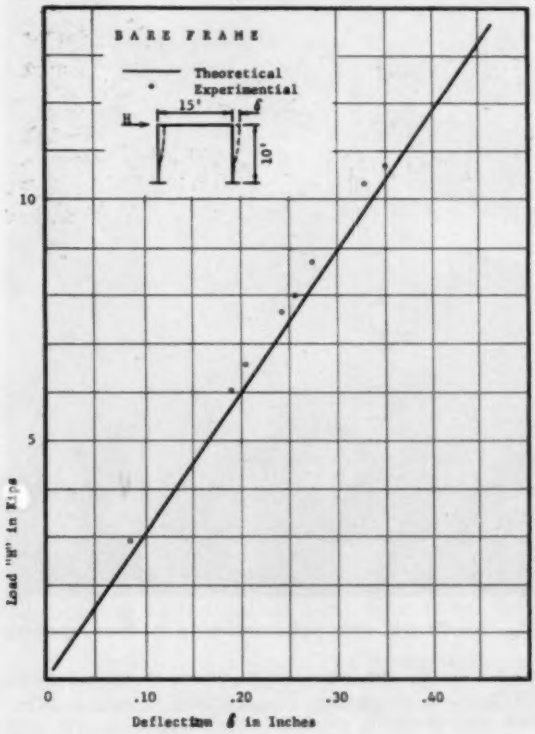

Fig. B—H-δ diagram for the bare frame

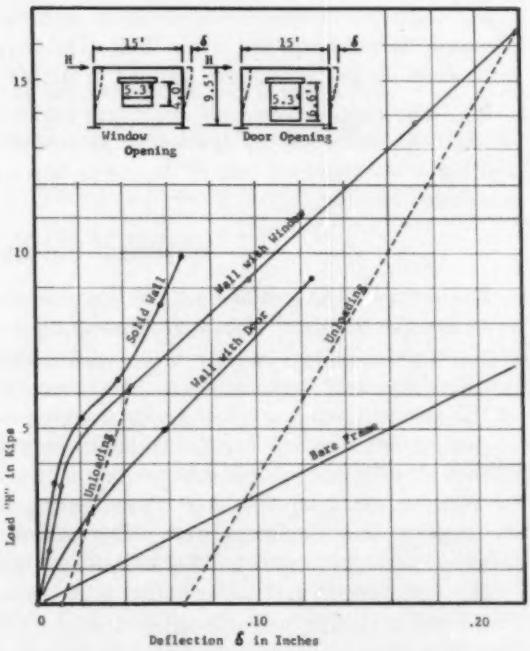

Fig. C — *H-δ* diagrams for various frame conditions

in which the stiffening effect was the same for both cases. This suggests that the wall had a pronounced damping effect on the deformation of the structure up to the first crack.

(3) For relatively high horizontal loads there was a plastic deformation in the frame which was greatest for the case of the frame with a wall containing a door. The residual stresses in the steel frame caused by the plastic deformation of the composite structure were relieved after the removal of the wall.

(4) The cracks in the walls containing openings followed the direction of a line passing through the point of application of the load and the upper left corner of the opening and another from lower right corner of the opening to the right column end.

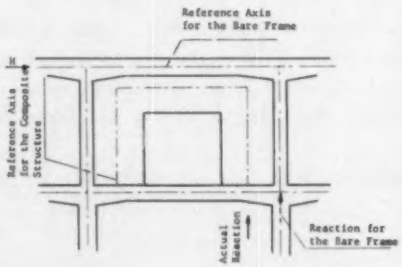
Reference Axis for the Bare Frame
H
Reference Axis for the Composite Structure
Actual Reaction
Reaction for the Bare Frame

Fig. D—Shifting of reference axis in a composite portal structure

(5) Each of the three walls tested showed a crack along the upper boundary between the steel and the wall. This type of cracking would probably occur only when the horizontal load applied to the structure was a concentrated one.

(6) The right reaction of the beam carrying the wall increased because of the application of the horizontal load. This behavior is explained by a shifting of the reference axis of the portal toward the interior of the composite structure (See Fig. D).

AUTHORS' CLOSURE

The authors are gratified by the interest shown in their paper and wish to thank the discussers for their comments.

Mr. Bertero is quite correct in his conclusion that the deflection at ultimate is poorly defined. Unfortunately, the behavior of a shear wall is a function of the testing technique once major cracking begins. In general, if the rate of loading is increased, higher strengths result with some change in the deflection at ultimate. When the loading is made in steps with time intervals for reading, cracking not only occurs during the actual loading but during the gage reading period as well. The authors felt that the tests were not extensive enough to give a reliable index of deflection at ultimate load.

The authors did test solid panels with vertical as well as shear loading. The vertical loading was applied to the panel edge rather than the columns. This vertical loading did not appreciably influence the shear wall behavior. Unfortunately, time allowed only limited testing and inasmuch as a definite trend did not appear, the study in this direction was abandoned.

The concrete block tests reported by Messrs. Cohen and Ozell are very interesting. The authors tested two panels similar to those tested by the discussers. One panel was solid and the other contained a window opening. The walls were reinforced with a steel ratio of 0.002 with typical reinforcing pattern. The steel appeared to have no influence on strength or deflection. The results of the tests were similar to those found by the discussers.

Discussion of a report by Subcommittee VII, ACI Committee 325:

Continuous Reinforcement in Highway Pavements*

By BENGT F. FRIBERG and SUBCOMMITTEE

By BENGT F. FRIBERG†

This first report of Subcommittee VII, Committee 325, includes items which may be clarified by discussion while continuously reinforced pavements are still subject to exploratory research, and before design criteria become standardized on the basis of observations and conclusions which are incomplete and, to some extent, controversial.

The essential characteristics of continuously reinforced pavements are: firstly, absence of closely spaced contraction or expansion joints; secondly, presence of longitudinal steel in sufficient amount and slabs of sufficient length so that closely spaced transverse cracks are inclined to occur, for distribution of length changes, which would otherwise occur only at widely separated points with much greater movements. There is no essential basis for the committee's suggestion that the design be limited to continuous pavements so long "that a considerable central portion is fully restrained against longitudinal movements due to seasonal temperature changes." The only essential functional design result are closely spaced cracks which might be counted on not to open wide enough at low temperatures to permit damage to the subgrade or to load transfer across the cracks. Considered on this basis continuously reinforced pavements could well include all appropriately reinforced unjointed pavements longer than conventionally reinforced slabs, which would be a more inclusive and logical scope of design, devoid of arbitrary limitation and changing interpretation.

Conventional plain pavements are built with slabs short enough to avoid significant cracking of the individual slabs. In conventional reinforced pavements cracks are anticipated as a normal occurrence, and it is intended that the steel should keep them from opening. In either type of design, 15 to 25 ft is accepted as a normal spacing between either joints or cracks that may occur. Cracks normally develop over several years. In appreciably longer slabs, combinations of traffic, load stresses, and curling or warping stresses may normally reach critical magnitude, although with favorable materials or environmental conditions in service and with light traffic, much longer slabs may remain intact or uncracked for much longer than usual periods.

*ACI Journal, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), p. 669. Disc. 55-42 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Consulting Engineer, St. Louis, Mo.

For the Indiana project, Cashell and Benham (*Proceedings*, Highway Research Board, V. 29, 1949, p. 68, Fig. 22) have demonstrated that frictional stresses have no significant influence on cracking of pavement slabs up to between 100 and 150 ft long, but that in longer reinforced pavements length has rapidly increasing influence on cracking, noticeable as greatly increased cracking frequency in central parts of longer slabs. Bar reinforced slabs approaching 300 ft in length exhibited crack spacing entirely characteristic for continuously reinforced pavements. For the steel percentages used, 0.45, 1, and 1.8 percent, the decreasing crack spacing in sections from 200 to over 1000 ft long was closely related to section length and independent of the amount of steel.

These sections in the Indiana project fit crack spacing, location, development, and other characteristics of continuously reinforced pavements, except the arbitrary minimum length for such pavements proposed in the report. It is suggested that observations on all 300 ft and longer sections of the Indiana project should be included in studies of continuous pavement behavior, especially since they contain steel percentages typical for continuous reinforcement and have reached a more appropriate age for appraisal than later projects.

The report quite correctly deals with some primary intended functional design details: causes of cracking, crack spacing and location, and crack width. Contrary to the expressed view in the report, reasons for cracking of continuously reinforced pavements appear to be relatively well established. Furthermore, the report seems to disregard the major variable bonding characteristics between the steel and concrete and their relation to early stress developments as well as continuing cracking and crack widths in continuously reinforced pavements.

REASONS FOR CRACKING

Occurrence of the characteristic cracking is generally ascribed to restraint tension stresses caused by frictional resistance to temperature contraction and shrinkage, with the latter influence pronounced in the drying concrete at early age and especially near the surface of the pavement. The maximum stresses are dependent on the magnitude or degree of restraints, higher in construction placed at comparatively high temperature and in concrete containing more steel with greater bond resistance at increased distances from free ends.

Closely spaced cracks are noticeable in high temperature construction even at early age, before traffic loads exert appreciable influence on stresses, indicating that contraction restraint and warping stresses together can reach critical magnitudes at early age. The flexural wheel load stresses and curling and warping restraint stresses increase gradually from cracks. Critical stress levels are reached by direct and flexural stresses in combination, although, as shown in published studies,* for steel with a high bond modulus tension

*Friberg, Bengt F., "Frictional Resistance under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs," *Proceedings*, Highway Research Board, V. 33, 1954, p. 167.

stresses alone can be critical without additive flexural stresses. Steel stress at crack and concrete stress between cracks are more dependent on the changing bond development between the steel bars and concrete than upon steel percentage. Design difficulties which have not been considered in the committee study are that bond slip can become localized to cause contraction predominantly at some cracks and greater than desirable movements at those cracks. The design objective of close crack spacing and narrow cracks can be attained with certainty only at early age because of progressive decrease in steel stresses at cracks with advancing age. It may involve uneconomical amounts of steel and is subject to uncertain variables of construction temperatures.

Restraint by the reinforcing steel to longitudinal curling is not believed to be a principal cause of cracking, although suggested in the report. It is inconsistent with behavior of cracked reinforced concrete that the reinforcing steel could "restrain the pavement from warping due to normal temperature gradients." The stiffness of the steel itself is insignificant and it could not influence materially that of the uncracked concrete between cracks. Any restraining couple at a crack could arise only as a result of opposing forces, or equivalent stress changes, on the concrete and the steel at the crack; but as both are generally subject to temperature variation in the same direction, substantial restraining couples could not occur normally. Curling restraints increase from ends over a comparatively short distance inward to a substantially constant maximum, related to the exposure, but would always be appreciably relieved at a crack. Incremental tension restraint stresses in the longitudinal steel coincide more closely with observed crack orientation, and are sufficient to explain the pavement behavior when considered together with normally occurring flexural stresses at early age.

CRACK SPACING AND CRACK WIDTH

Any pavement restrained against length change must undergo a stress change, the strains for which constitute the restraint to length change. Full restraint to temperature drop in concrete equals 10 to 25 psi per deg F temperature change. Obviously concrete cannot sustain too great a temperature drop, or its equivalent in shrinkage, without critical tension stress. When cracks occur to relieve concrete tension at the crack, tension can be reintroduced into the concrete only through bond on each side of the crack. Steel stresses at cracks, and partial concrete restraint stresses after cracking, are primarily gaged to steel bonding properties. Normal steel bonding characteristics involve sufficient slip for primary adjustments to low temperatures to take place in the form of elongation of the steel across the crack for some distance on each side, visible as crack widening, rather than as elongation of the concrete between cracks under tension restraint stress. Crack widths would then be closely related to crack spacing primarily, and not the the concrete stress imposed by the steel.

Maximum stress at cracks and extent of active bond on each side occur at the lowest temperatures in pavements under full restraint. The maximum crack widths are nearer to values based on temperature contraction and moisture change than to "infinitesimal width changes." Bond slip being a nonelastic property, cracks remain open to some extent subsequently, unless closed by compressive stress in the steel. Soil infiltrates into the cracks at low temperature so that actually they do not close even at high temperatures, subjecting the pavement to substantial compressive stress.

The report refers to width measurements on the Illinois 7- and 8-in. pavements at 10 years' age, which averaged 0.034, 0.023, 0.016, and 0.010 in. for average crack spacings of 12, 8, 6, and 5 ft in pavements reinforced with 0.3, 0.5, 0.7, and 1.0 percent steel, respectively. These crack widths were measured at 106 F slab temperature, and it is doubtful that actual widths below the surface were any less at the conditions of compressive restraint which must have existed, a condition different from that suggested in the report, which would be typical for much lower temperatures. The total crack widths indicated by the measurements are:

For 0.3 percent steel, 12-ft spacing, crack width = 0.28 in. per 100 ft
For 0.5 percent steel, 8-ft spacing, crack width = 0.29 in. per 100 ft
For 0.7 percent steel, 6-ft spacing, crack width = 0.27 in. per 100 ft
For 1.0 percent steel, 5-ft spacing, crack width = 0.20 in. per 100 ft

(The average crack widths were based on measurements of 60 cracks in five groups, which gives a more representative integrated value for large than for small crack spacing.)

The data show surprising uniformity of total crack width for 0.3 to 0.7 percent steel, indicating that the crack widths cited in the report, for those steel percentages at least, is purely a function of crack spacing. The data do not support suggestions in the report, that the reinforcing steel "controls the amount of crack opening *in addition* to governing the crack interval" (italics are those of this discusser) and "that there be enough steel to prevent any excessive opening of the cracks."

Contrary to expressed design objectives, it would appear from the Illinois measurements as well as measurements on the New Jersey project,* data from which are shown in Fig. A, that cracks open appreciably at low temperatures and thereafter remain open to some substantial degree. The reason must be that soil does infiltrate and remains in the cracks; however, possibly because of compression across the infiltrated cracks at increasing temperature, the aggregate interlock and effective sealing may not be lost. The observed crack widths at high temperature on the Illinois project and in the central fully restrained portions of the New Jersey project indicate the presence of substantial compression restraint stresses with advancing age at high temperature. Variations in crack width at low temperatures, without compression restraints across cracks, have not been explored adequately on

*Van Breemen, William, "Report on Experiment with Continuous Reinforcement in Concrete Pavement—New Jersey," *Proceedings*, Highway Research Board, V. 30, 1950, p. 61.

existing projects. General observations indicate that adequate load transfer and sealing have existed across cracks which have been wider than generally recognized limits for open cracks. Maximum acceptable crack spacings and crack widths for the intended design objective have not been established. Crack spacings of 8 to 10 ft appear to be acceptable, provided occasional wide cracks could be avoided.

The primary difference in behavior between conventional short slab construction and continuously reinforced pavements would be the shorter components between cracks in the latter when built at high temperature, correspondingly narrower cracks at low temperature, more uniform in width

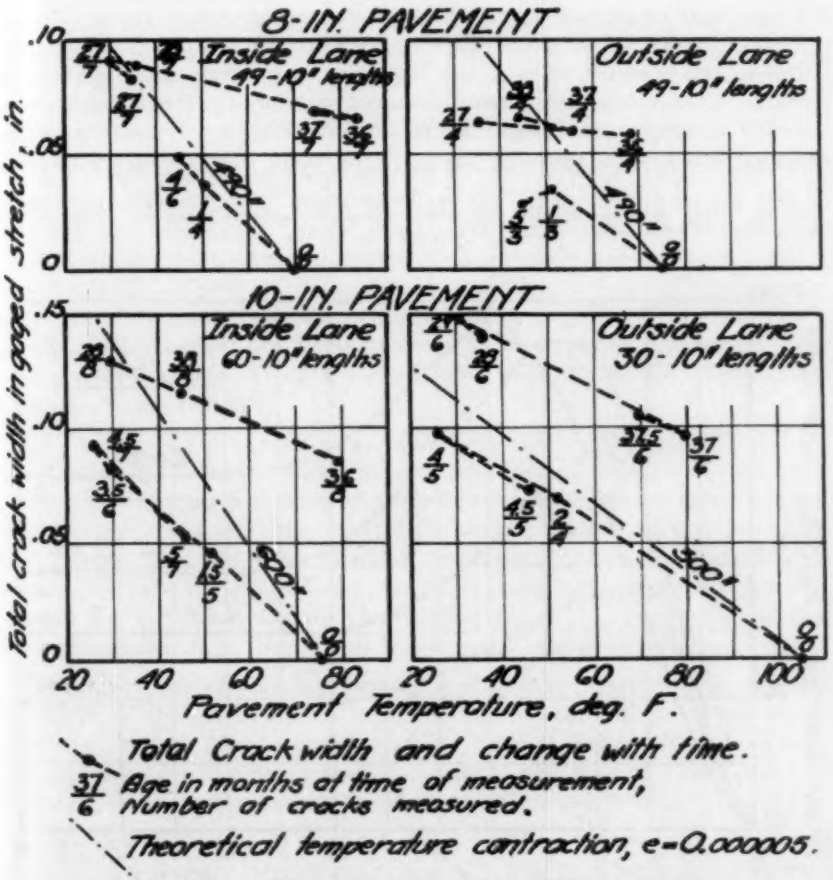

Fig. A—Total crack widths in relation to slab temperatures, based on 10-in. gage measurements across crack in fully restrained locations on the New Jersey continuously reinforced pavement. Crack widths for unrestrained temperature contraction of the concrete are indicated

because of the ties across them, and load transfer preserved across the narrower uniform crack widths, irrespective of soil infiltration. Plastic flow in tension at low temperature is not believed to be a factor influencing crack width. Compression is much more prevalent, and creep at high temperature restraint may be a factor in minor progressive increase in crack widths.

Relations between restraint conditions and crack occurrence are well illustrated on the Illinois project. Fig. B shows variations in cracking inward from the free ends to pavement sections under full restraint at all seasons, based on the 10-year report.* The crack frequency in central parts is substantially lower than the maximum which occurred 200 to 600 ft from the ends. The different sections expanded seasonally and progressively the full 4-in. available expansion space between the ends in about 5 years. Tension restraint stresses would therefore occur with but small decrease in temperatures in the portions where crack frequency is greatest, while the central portions would remain under compression restraint for even lower temperature, reflected in greater crack spacings in central portions. Substantial crack widening and tension restraint stresses could occur only below compression

*Lindsay, J. D., "A 10-year Report on the Illinois Continuously-Reinforced Pavement," *Bulletin* No. 214 Highway Research Board, 1958.

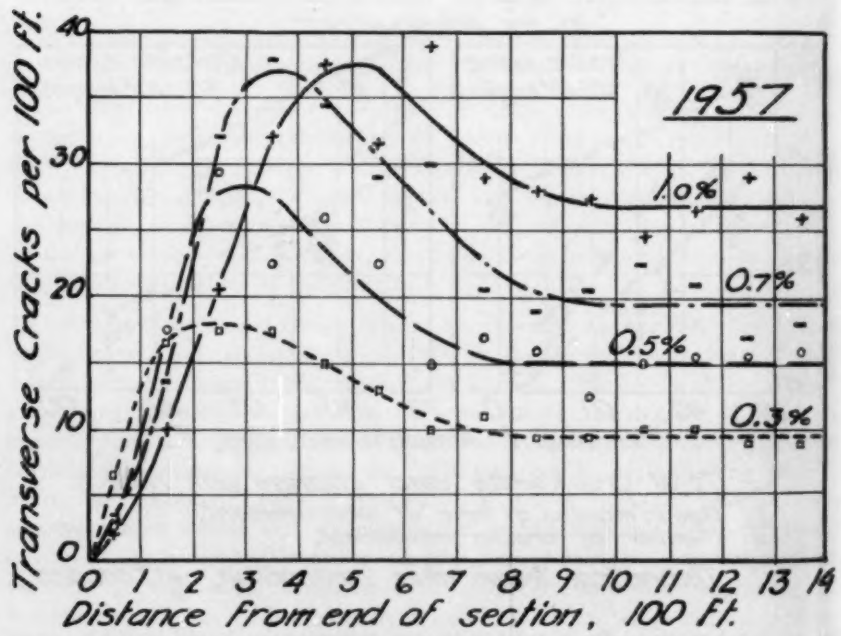

Fig. B—Frequency distribution at 10 years age of transverse cracks in relation to distance from the ends on 7-in. continuously reinforced pavement sections in Illinois, reinforced with 0.3, 0.5, 0.7, and 1 percent longitudinal steel

restraint temperatures. Because of crack infiltration and physical changes in the concrete, compression restraints may exist at progressively lower temperature with advancing age.

A clear understanding of pavement restraints, and eventual economical design of continuous pavements, will come from careful study of movements, real crack widths, and steel stresses, considering the narrow cracks near unrestrained ends, those in intermediate sections restrained for daily but not seasonal movements, and those in central fully restrained sections, as interrelated but separate behaviors, but all within the scope of continuously reinforced pavements. Shrinkage and expansion phenomena, warping slopes and gradients cannot be disregarded in observations if we wish to explain adequately the behavior of continuous pavements. The great material costs involved cannot be justified on only superficial observations and deductions.

SUBCOMMITTEE CLOSURE

The authors appreciate the discussion by Mr. Friberg. In preparing its report the subcommittee, realizing that continuous reinforcement was a relatively new subject, purposely pointed out the areas in which knowledge was lacking and the conjectural nature of some of its reasoning so as to stimulate discussion. There are a number of interesting aspects of the behavior of continuously reinforced pavements for which complete theoretical explanations are lacking. Until adequate research is completed, the explanations are merely reasonable hypotheses which agree generally with research results which are now available.

Mr. Friberg has presented some interesting and worthwhile explanations for the observed behavior of continuously reinforced pavements which should be helpful in the further study of the subject. As he states, direct tension unquestionably is a principal cause of the development of transverse cracks. It is possible that the same cracking frequency would occur eventually if warping and load stresses were not present, but undoubtedly load and warping stresses are important factors contributing to crack development and it seems reasonable that they would increase the rate at which cracks occur.

The importance of warping stresses is shown by some notable research by the U. S. Bureau of Public Roads.* Fig. 38, 39, and 40 of this report show that warping stresses may be of the order of 300 to 400 psi. This is also confirmed by Fig. 19 of the same report† and by Fig. 151 of the final report on Road Test 1MD published by the Highway Research Board.

Teller and Sutherland* (p. 194) state: "The stresses from this cause (warping) are actually large enough to cause failure in concrete of low flexural strength, and since the direction of stresses is such that they become added

*Teller, L. W., and Sutherland, E. C., "Structural Design of Concrete Pavements. Part II," *Public Roads*, V. 16, No. 9, Nov. 1935.
†Teller, L. W., and Sutherland, E. C., "Structural Design of Concrete Pavements. Part III," *Public Roads*, V. 16, No. 10, Dec. 1935.

to critical stresses caused by wheel loads, there is little doubt but that warping stress is primarily responsible for much of the cracking in concrete pavements."

Mr. Friberg calls attention to the uniformity of accumulated crack width per hundred feet of slab found in the Illinois project and states that, "the data do not support suggestions in the report, that the reinforcing steel controls the amount of crack opening in addition to governing the crack interval." Regardless of the fact that the accumulated crack width is the same for the sections containing 0.3, 0.5, and 0.7 percent steel, it is nevertheless true that the average crack width varies inversely with the amount of steel. The present condition as regards crack width, of the Illinois sections containing 0.3 percent steel, shows conclusively that a greater amount of steel is desirable for proper performance.

Mr. Friberg by reference to data from the Indiana project concludes that crack frequency is closely related to section length and independent of the amount of steel. It should be remembered that in this project the amount of steel was by design related to the section length. Furthermore, in the Illinois projects where six of the eight sections were of equal length, there is a definite relationship between crack frequency and amount of steel.

It is admitted that the definition for continuously reinforced pavement as stated in the subcommittee report is arbitrary, and Mr. Friberg is theoretically correct in his objection to assigning a minimum length to continuously reinforced pavement. But it would seem that from a practical point of view this is necessary, since the only real justification for such construction is that long stretches of pavement can be built without joints. If, as Mr. Friberg suggests, the design were applied to relatively short slab lengths, jointing problems of serious proportions would be introduced. The expansion joints in the Illinois projects at 3500- and 4200-ft intervals have contributed to pavement distress and created serious maintenance problems.

The present report of Subcommittee VII is clearly a preliminary evaluation of available knowledge on the subject, and it is hoped that it will provide a guide for future research and for the design and construction of other projects. If it stimulates discussion such as that offered by Mr. Friberg it will have fulfilled a useful purpose and will contribute to the advancement of knowledge in the theory of continuous reinforcement and to improved design and construction practices and techniques.

Discussion of a paper by James S. Blackman, Gerald M. Smith, and Lyle E. Young:

Stress Distribution Affects Ultimate Tensile Strength*

By PER K. BREDSDORFF† and P. KIERKEGAARD-HANSEN‡

The authors have presented test results indicating that stress distribution has an appreciable effect on the ultimate tensile strength of concrete mortar specimens.

Such effects may be predicted from statistical theories of the strength of materials. Readers interested in this subject are referred to a paper by Johnson[5] which contains extensive references to the literature and a rigorous mathematical development of these theories and their application to engineering problems.

The writers have investigated whether the pattern of the test results are in agreement with Johnson's formulation of the statistical theory of the strength of perfectly brittle materials, i.e., materials with the property that total failure occurs instantaneously when, without previous plastic deformations, fracture occurs at a single point. The conclusion of this investigation is that the test results exhibit a greater increase of ultimate tensile strength, when the volume exposed to tension is reduced, than might be expected from the statistical theory applied to the conditions of the tests.

It is possible that a modification of the statistical theory, more suitable to the nature of concrete and its mechanism of failure, might result in better agreement.

The degree of nonhomogeneity of the material is the starting point of the statistical theory. In the theoretical model this feature is accounted for by assuming that a specimen with unit volume, 1 x 1 x 1 in. is built up by a great number (n_1) of equally small elementary volumes, differing in ultimate tensile strength. (The ultimate strength of the elementary volumes in other directions than that of tension may safely be neglected in the case of concrete subjected to tension.) The n_1 elementary volumes are considered to be a random sample from an infinite number of such volumes, characterized by a distribution function, $F(x)$. $F(x)$ indicates the probability that an elementary volume has ultimate tensile strength less than or equal to a specified value, x. $F(x)$ is zero for negative values of x; from $x = 0$ the distribution function is a never decreasing continuous function of x. $F(\infty) = 1$.

*ACI JOURNAL, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), p. 679. Disc. 55-43 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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‡Assistant Professor, Danish Engineering Academy, Copenhagen, Denmark.

If a beam of dimensions l x b x d is subjected to uniform tensile stress produced by axial loading, the probability of failure at an ultimate tensile strength less than or equal to x will be

$$G(x) = 1 - [1 - F(x)]^n,$$

where n, the number of elementary volumes, is equal to $l \cdot b \cdot d \cdot n_1$. $G(x)$ indicates the probability for the smallest value (weakest point) in a sample of n values to be less than or equal to x.

If specimens of dimensions l x b x d are made according to the same specifications and subjected to uniform tension it is to be expected that the test results will be distributed according to $G(x)$.

Under certain conditions it may be demonstrated[5] that when n_1 and n are great numbers, $G(x)$ tends to an exponential function which may conveniently be written in the form

$$G(x) = 1 - e^{B}; \qquad B = -\frac{n}{n_1}\left[\frac{x}{m_1}\,\Gamma\left(1 + \frac{1}{k}\right)\right]^k$$

where m_1 is the mean value of the ultimate tensile strength of specimens with unit volume, i.e., consisting of n_1 elementary volumes, and k is a constant depending of the form of $F(x)$.

The mean value of ultimate tensile strength of specimens of volume $lbd = n/n_1$ is

$$m = m_1 \left(\frac{n_1}{n}\right)^{\frac{1}{k}}$$

and the coefficient of variation, σ/m, is

$$\frac{\sigma}{m} = \frac{\sigma_1}{m_1} = \sqrt{\frac{\Gamma\,(1 + 2/k)}{\Gamma^2(1 + 1/k)} - 1}$$

It is a remarkable consequence of the theoretical assumptions that the coefficient of variation is independent of n/n_1, the size of the volume. The implied relation between k and σ/m is illustrated on the two abscissae-scales on Fig. A. It should be borne in mind, that the constancy of σ/m for given k presupposes that the lowest possible value of x is zero.

When the beam is subjected to the loading systems numbered b to f in the authors' paper, the stress distribution in the beam is not constant and this fact must be accounted for in the calculations leading to the probability distributions of the tensile strength according to the "weakest point" theory.[5,6]

The tensile stress, x_{uv}, in every point of the u-v plane for beams with rectangular cross section, see Fig. B, may be expressed in the form

$$x_{uv} = xh(u,v)$$

where $h(u,v)$ is a function which takes on the value 0 at the neutral axis and the value 1 at the extreme tension fibers in the region of constant moment. Besides, $h(u,v) = 0$ in all compressive fibers. The distance from the neutral axis to the extreme tension fiber is d' in the region of constant moment.

The exponent B in the distribution function for the ultimate tensile strength, measured by x, is then obtained as b times the double integral of the function

$$\left[\frac{xh(u,v)}{m_1}\,\Gamma\left(1+\frac{1}{k}\right)\right]^k$$

evaluated over that part of the area in the u-v plane, which is subjected to tension.

By carrying out these integrations for the six loading systems considered by the authors you get the following results:

Loading system	The exponent, B, in the expression, $G(x) = 1 - e^B$, for the distribution function of the ultimate tensile strength.
a	$B_a = -lbd\left[\frac{x}{m_1}\,\Gamma(1+1/k)\right]^k$
b	$B_b = -\frac{lb}{k+1}\left[d' - \frac{(d'-d)^{k+1}}{(d')^k}\right]\left[\frac{x}{m_1}\,\Gamma(1+1/k)\right]^k$
c	$B_c = -\frac{2b}{(2ld')^k\,(k+1)}\left(\int_0^{u_c}\frac{-1}{6u}\,(2ld' - dl - 3ud)^{k+1}\,du + \int_0^{l/3}\frac{1}{6u}\,(2ld' - dl + 3ud)^{k+1}\,du + \frac{1}{12}\,(2ld')^{k+1}\right)\left[\frac{x}{m_1}\,\Gamma\left(1+\frac{1}{k}\right)\right]^k$
d	$B_d = -\frac{lbd'\,(1+k/3)}{(k+1)^2}\left[\frac{x}{m_1}\,\Gamma(1+1/k)\right]^k$
e	$B_e = -\frac{2b}{(2ld')^k\,(k+1)}\left(\int_{u_c}^{l/3}\frac{1}{6u}\,(2ld' - dl + 3ud)^{k+1}\,du + \frac{1}{12}\,(2ld')^{k+1}\right)\left[\frac{x}{m_1}\,\Gamma\left(1+\frac{1}{k}\right)\right]^k$
f	$B_f = -\frac{lbd'}{(k+1)}\left[\frac{x}{m_1}\,\Gamma(1+1/k)\right]^k$

Note: Formulas for B_a, B_d, and B_f are given in Reference 5 and 6.

From the distribution functions, the corresponding mean values are evaluated as indicated below (suffix letters indicate loading system)

$$m_a = m_1 \sqrt[k]{\frac{1}{lbd}}$$

$$m_b = m_1 \sqrt[k]{\frac{k+1}{lb\,[d' - (d' - d)^{k+1}/(d')^k]}}$$

$$m_c = m_1 \sqrt[k]{\frac{[(x/m_1)\,\Gamma(1+1/k)]^k}{-\,B_c}}$$

$$m_d = m_1 \sqrt[k]{\frac{(k+1)^2}{lbd'\,(1+k/3)}}$$

$$m_e = m_1 \sqrt[k]{\frac{[(x/m_1)\,\Gamma(1+1/k)]^k}{-\,B_e}}$$

$$m_f = m_1 \sqrt[k]{\frac{k+1}{lbd'}}$$

For the specimens tested by the authors the relevant values of the dimensions are: $b = d = 1$; $l = 6$ for loading systems a, b, and c; $l = 9$ for loading systems d and e; and $l = 11$ for loading system f.

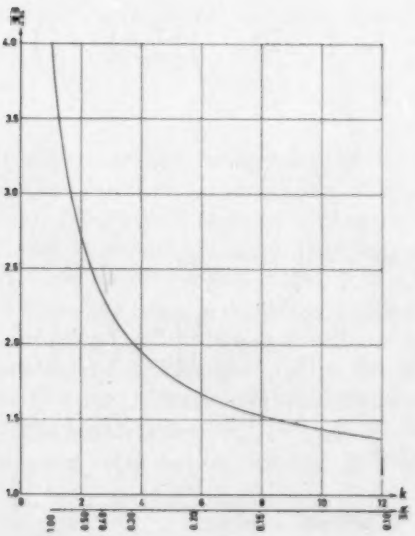

Fig. A—m_d/m_a as function of k and the coefficient of variation, σ/m.

$$\frac{m_d}{m_a} = \sqrt[k]{\frac{l_a bd\,(k+1)^2}{l_b bd'\,(1+k/3)}} = \sqrt[k]{\frac{4\,(k+1)^2}{3\,(1+k/3)}}$$

on the assumption that $l_a = 6$, $l_b = 9$, $b = d = 1$, $d' = \frac{1}{2}$

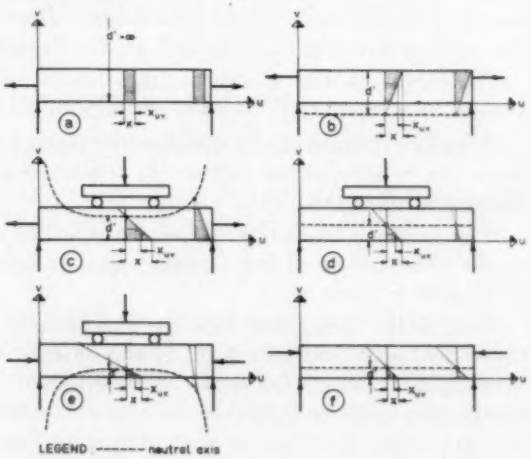

Fig. B—Coordinate systems, (u,v), for stress distributions corresponding to the authors' loading systems, a-f.

It will be seen that when k is known the theory implies definite relationships between the curves for the mean values, considered as functions of d'.

To test the theory we have to estimate k from the observations reported by the authors. Fig. *A* is drawn to illustrate the implications of such an estimate. We have estimated $k = 8$ which implies $\sigma/m = 0.148$ and $m_d/m_a = 1.58$. The test results indicate that the coefficient of variation is slightly less than 0.148 but it may be considered as an admissible value.

In the interpretation of the test results we have assumed that ultimate tensile strength is proportional to ultimate tensile strain, the proportionality

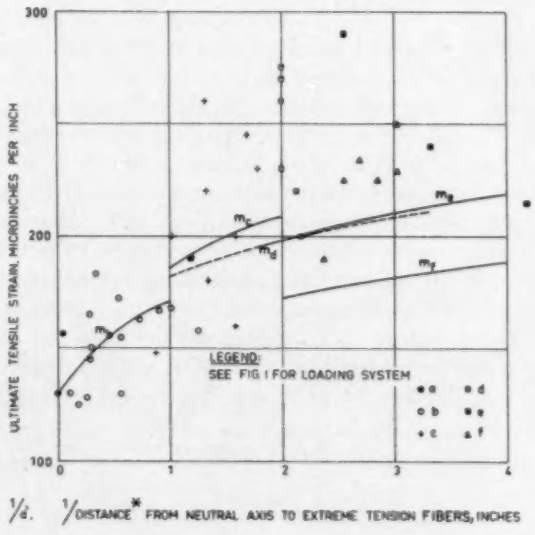

Fig. C—Effect of strain distribution on the ultimate tensile strain. (Test results reported by the authors converted to the scale $1/d'$ as a measure of strain distribution)

*in the region of constant moment.

factor being the same for all specimens. The test results reported in Fig. 4 are indicated in Fig. C with $1/d'$ as the abscissa.

The curves on Fig. C indicate the theoretical mean values for the loading systems *b*, *c*, *d*, *e*, and *f*, on the assumption that $k = 8$ and, $m_a = 130$.

It will be seen that the observed increase in ultimate tensile strain with reduction in the volume exposed to tension is greater than predicted by the theoretical relations.

If m_a is given an estimated value above 130 the curves for *d*, *e*, and *f* will fit the observations slightly better, but at the same time the curves for *b* and *c* will give a worse fit.

Even if the agreement between theory and observation is imperfect the theoretical considerations may, in the writers' opinion, be some of value in drawing attention to the degree of homogeneity of the specimens—measured by the coefficient of variation—as a possible important factor in the relation between strain distribution and ultimate tensile strain. As the coefficient of variation is a characteristic of the test conditions this possible influence should be borne in mind before generalizations from the observed relationship are drawn.

REFERENCES

5. Johnson, Arne, "Strength, Safety and Economical Dimensions of Structures," Stockholm, 1953.

6. Weibull, W., "The Phenomenon of Rupture in Solids," *Proceedings*, No. 153, The Royal Swedish Institute for Engineering Research, Stockholm, 1939.

Discussion of a paper by E. M. Rensaa:

Shear, Diagonal Tension, and Anchorage in Beams*

By JAMES CHINN, BRUCE H. FALCONER, HANS GESUND, K. HAJNAL-KÓNYI, MAX HERZOG, A. M. NEVILLE, R. B. L. SMITH, and AUTHOR

By JAMES CHINN†

Mr. Rensaa has compiled the results of much research both European and American, of his own experience and analyses of structures, in a thought-provoking article for which he is to be highly commended. The writer wishes to add some of his own experiences in support of several of the points brought up by Mr. Rensaa.

The writer is in full agreement with the author, that laboratory investigations do not reflect the effect of shrinkage. Mr. Rensaa presents a calculation indicating that shrinkage can cause a compressive stress of 7533 psi in reinforcing steel. Rouse,[23] in tests performed at the University of Colorado, installed reinforcement gages in longitudinal reinforcing, vertical stirrups, and bent-up bars in beams. Initial or zero readings of these gages were taken immediately before the beams were cast. The stresses in steel in unloaded beams caused by shrinkage were determined over a long period. For practically all strain meters, it was found that the stress in steel increased to a value of approximately 8000 psi compression at 28 days after casting. (Beams were moist-cured for 8 days then allowed to sit with no covering in laboratory air thereafter.) It was also found that vertical stirrups did not take load until they were crossed by a diagonal crack.

The writer has witnessed an instance where a girder framed to a continuous column, as in Fig. A, received its load for a part of its length from pan joists framing to one side only. For a short length adjacent to the column there were no pan joists and the girder was a rectangular beam rather than a T-beam. For design purposes the girder was assumed essentially, to be simply supported at the column, so half the bottom steel, three #10 bars, were bent up at one-seventh the clear span. The additional three #10 bottom bars were carried into the column, and #3 stirrups were provided for shear. Due to a combination of shear, axial tension due to shrinkage, and torsion due to load from one side only, a diagonal crack formed as shown under dead load alone. The crack appears to run the full depth of the girder. This girder complies in every way with the requirements of the ACI Code, and the writer believes that it is structurally adequate. This would appear to be essentially the

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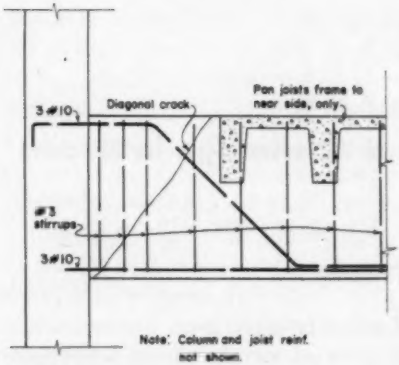

Fig. A—Diagonal crack formed by shear, tension, and torsional stresses

same condition as at a point of contraflexure, and even the bent-up bars, vertical stirrups, and longitudinal steel were not effective in preventing formation of a diagonal crack. There were four locations in this building where this same structural condition existed and the diagonal crack formed at all four. At the opposite ends of these same four girders, the pan joists extended to the columns so the girders were flanged on one side. No diagonal cracks formed at these locations although subjected to greater shear.

In all tests known to the writer where stirrups or diagonal bars were used as web reinforcing, the web reinforcing was ineffective in preventing diagonal cracking even though effective in increasing ultimate shear capacity. Tests now being performed by the writer indicate that this is also true of longitudinal bars placed at the middepth.

It is hoped that researchers will give serious thought to Mr. Rensaa's paper and bear it in mind in planning future tests.

REFERENCE

23. Rouse, George C., "A Method for Determining Stresses in Concrete Reinforcement During Long-Time Tests," *Technical Memorandum* 655, U. S. Department of the Interior, Bureau of Reclamation; Sept. 1958, 82 pp.

By BRUCE H. FALCONER*

The discusser is in full agreement about the importance of the following oft-neglected points;

(a) Both compressive and tensile strengths of concrete should be strictly considered.

(b) Allowance should be made for possible longitudinal forces.

(c) Concentrated load from a support may reduce principal tensions locally to a significant extent.

(d) Bent-up bars and inclined stirrups are, in general, more efficient than vertical stirrups; particularly at points of contraflexure.

(e) It is useful, in the deeper beams at least, to include some intermediate longitudinal reinforcement.

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(f) Some minimum number of stirrups in all beams of ordinary size and importance is advantageous.

(g) Caution should be used so as not to cut bars too short when stopping them off according to the flexural moment.

However he wishes to dispute the contention that at points of contraflexure web reinforcement, when required, should necessarily be adequate to carry all the shear according to conventional theory of stresses in stirrups.

Interest in this particular aspect is heightened by the thought that in seismic regions (the New Zealand code, for instance, requires 6, 8, or 10 percent of gravity base shear resistance in buildings) there are circumstances in which points of contraflexure may move anywhere in the length of a member.

The divergence of opinion hinges on the discusser's belief that for attainment of a given shear resistance in a given member both the primary (longitudinal) and secondary (web) reinforcement must be adequate, but the relative proportions may be safely varied within limits. This belief involves the contention that in reinforced concrete the reinforcement need not be provided purely to react the principal stresses of the applied stress field (i.e., the principal tensions that would exist in the concrete if unreinforced, uncracked, and homogeneous). Indeed the presence of reinforcement, and the stresses in and on it, must, in the general case, radically alter the stress system in the concrete; yet the resultant of the two induced component stress systems, the one in the reinforcement and the other in the concrete, must remain equivalent to the applied stress field.

The simplest qualitative illustration of this fact may be had by considering the various patterns of reinforcement suitable for strengthening a wall panel which is subject to a uniform shear traction around its boundaries inducing shear.

1. Should the reinforcement be provided solely in the direction of the principal tension of the applied field the concrete would be required to carry only the pure compression in the direction of the compressive principal stress of the applied stress field.

2. Should the same reinforcement be provided but in a direction making a slight angle with that of "principal tension" then the concrete is required to carry compression of about the same magnitude as the "principal compression" but in a slightly altered direction, plus a slight tension.

3. Should the theoretician elect to proclaim that concrete "can carry no tension" (he must still recognize the essential dependence of bond and concrete cover on tensile ability) then a small quantity of reinforcement must be added in a second direction, for Case (2), so that the concrete is again required to carry only pure compression (though it will suffer stress less tensile extensions in another direction). Once again the compression in the concrete would be altered slightly in both magnitude and direction.

The discusser considered such a problem[24] and showed, to his own satisfaction at least, that the quantities of reinforcement, in any two arbitrary directions, required to provide ultimate resistance can be computed. As a particular case it was shown that

4. Should the reinforcement be placed in two orthogonal directions at 45 deg to the principal tension then, (a) if the quantities of reinforcement were equal each carried forces of magnitude equivalent to the applied shear stress, and the concrete was required

to sustain a compression of twice the principal compression, in the direction of principal compression, but (b) if the quantities of reinforcement were unequal then the compression in the concrete would be greater than twice the principal compression and at an angle with it.

Quantitatively, it was shown in the case of 4(b) that

$$\sqrt{\sigma_x \sigma_y} = v, \quad \text{and} \quad \sigma_y > \frac{v^2}{f_c' - \sigma_y} \qquad (1)$$

The closer σ_x is to σ_y the greater is the economy in weight of reinforcement, and the greater the attainable shear strength before crushing of the concrete.

For general design of shear walls the discusser is content to provide ratios of reinforcement area to concrete area given by,

$$\frac{1}{\sqrt{2}} \frac{v}{f_s} < p_y < \sqrt{2} \frac{v}{f_s}, \text{ and } \sqrt{p_x p_y} = \frac{v}{f_s} \qquad (2)$$

where

v = applied shear stress
σ_x = equivalent stress of primary reinforcement
σ_y = equivalent stress of secondary reinforcement
f_c' = crushing strength of concrete
p_x = ratio of area of primary reinforcement
p_y = ratio of area of secondary reinforcement
f_s = yield strength of reinforcement

The proportions are selected for limiting strength.

The extension of this argument to beams was given in an appendix to Reference 24, where it was shown that for beams with vertical web reinforcement "tension flange" reinforcement should be provided to carry a force of

$$F = \frac{M}{jd} + \tfrac{1}{2} V \beta \qquad (3)$$

and stirrups to carry tensions equivalent to a stress of

$$\sigma_y = \frac{1}{\sqrt{\beta}} \frac{V}{bjd} \qquad (4)$$

where $$\beta < \frac{v^2}{\sigma_y^2} = \frac{f_c'}{\sigma_y} - 1$$

and M = applied moment at section considered
V = applied shear at section considered
jd = effective lever arm of section
b = breadth of member

Should the design be in terms of normal working stresses, rather than limiting stresses, then the appropriate minimum web reinforcement should be computed using β from

$$\frac{f_c'}{K\sigma_v} - 1 > \beta > 1$$

Should β be taken greater than 1 ($\sqrt{2}$ is suggested for higher loadings) greater attention than usual should be paid to the quality and anchorage of longitudinal reinforcement, and to the distribution and anchorage of web reinforcement.

For inclusion of formulas in codes, where equations should be simple, the discusser would compromise somewhat. He suggests the following.

"In beams having vertical web reinforcement longitudinal reinforcement shall be adequate to carry flange tensions of

$$F = \frac{M}{jd} + \frac{V}{2} \quad \text{.....(3a)}$$

and web reinforcement shall be adequate to carry forces equivalent to a stress of

$$v_s = \frac{v^2}{0.10 f_c'} \quad \text{.....(4a)}$$

where $v = \dfrac{V}{bjd} < 0.11 f_c'$. The minimum ratio of web reinforcement shall be

$$p_v = \frac{v_s}{f_v} = \frac{v^2}{0.10 f_v f_c'} \quad \text{.....(5)}$$

f_v = tensile unit stress in web reinforcement

Where beam ends are supported by beams of similar depth the minimum value of v_s within a distance of d from the junction shall not be less than $v/\sqrt{2}$."

A comparison of the minimum stirrup reinforcement required by Eq. (4a) with that for the current ACI, British (BSCP), and New Zealand (NZSS) codes is shown in Fig. B for f_c' of 3000 psi.

It is obvious that any theory is limited by the aptness and validity of the initial assumptions. Because boundary effects of stirrup anchorages and stirrup spacings and covers to reinforcement are ignored, it is not surprising that tests indicate that at the higher shears the full theoretical strengths are not attained. This fact influences the choice of 0.10 as a parameter in the arbitrarily amended Eq. (4a). For f_c' over 3000 psi the parameter could perhaps well be taken as $0.10\,[1 - \frac{1}{2}\,(f_c' - 3000)/f_c']$.

The discusser would like to reiterate the suggestion given earlier that it would be of great interest if a series of test beams be loaded as in Fig. C. Such a system of loading would obviate the presence of local arching within

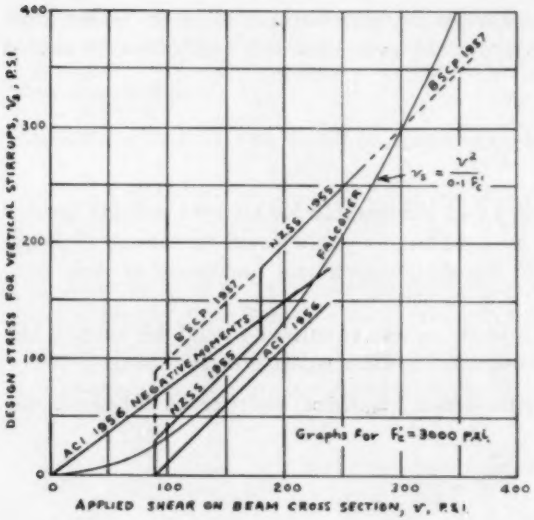

Fig. B—Minimum stirrup reinforcement required by ACI, British, and New Zealand codes

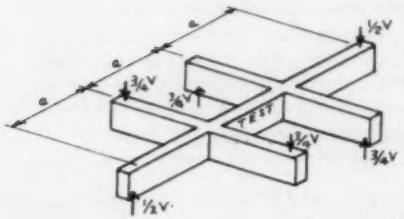

Fig. C—Advocated test load system for applying shear to obviate local arching within test length

the test length, due to vertical compression from forces applied at upper and lower surfaces of a test length. It could well throw new light on the problem of shear at points of contraflexure.

REFERENCE

24. Falconer, Bruce H., "Theory of the Stresses Induced in Reinforced Concrete by Applied Two-Dimensional Stress," ACI JOURNAL. V. 28, No. 3, Sept. 1956 (*Proceedings* V. 53), pp. 277-293.

By HANS GESUND*

Much gratitude is due Mr. Rensaa for his clear and comprehensive review of the basic problem and past history of shear and diagonal tension in reinforced concrete members. The writer was particularly struck by Mr. Rensaa's presentation of the problem in continuous members, since, after all, most reinforced concrete members are to some extent continuous. The writer

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believes, however, that a few minor points may still be added to the author's presentation.

First is the problem of the participation of the concrete in carrying the shear. It seems anomalous to expect the concrete to take diagonal tension in shear, when it is assumed that it cannot take tension in bending. There is the usual justification that the diagonal tensile stresses are much lower than the bending stresses, but on the other hand, the concrete must crack long before the shear reinforcement can reach its design stress. As Mr. Rensaa has pointed out, if the point on the beam in question is a point of zero bending moment, there will be no compressive stress block, and hence, if the stirrups are stressed to their design stress, they must carry the entire shear. However, it should be noted that the presence of bending, and hence of a compressive stress block, does not materially alter the situation. A combination of compressive stress in one direction, zero stress in the direction perpendicular to the compressive stress, plus shearing stress, will result in a principal tensile stress. This stress will be lower than for the case of pure shear, and any crack that may form in that region will be more inclined toward the horizontal, nevertheless, the tensile stress still exists and is more safely carried by steel than by concrete. For this reason, the writer believes that the shear reinforcement should be so proportioned as to be able to carry the full shearing force.

This leads directly to the next point, the method of proportioning the reinforcement and the relative efficiency of vertical and inclined stirrups. The writer has found the inverted keystone concept to be a valuable visual aid in examining the effects of shear in concrete beams. Fig. D represents a portion of a continuous beam with all loads acting on it represented by P. Assuming, for now, that the critical sections are at the inflection points, failure would occur when the diagonal tensile cracks formed, the strirups failed, and the inverted keystone dropped out of the structure. Assuming further that the diagonal tension crack would be inclined at an angle Θ to the horizontal consistent with the direction of the principal tensile stress, and also assuming that the jd depth of the beam is the effective length of the shear reinforcement, the condition of vertical equilibrium would require that $A_v f_v jd/s = V \tan \Theta$ where the symbols have the same meaning as in (ACI

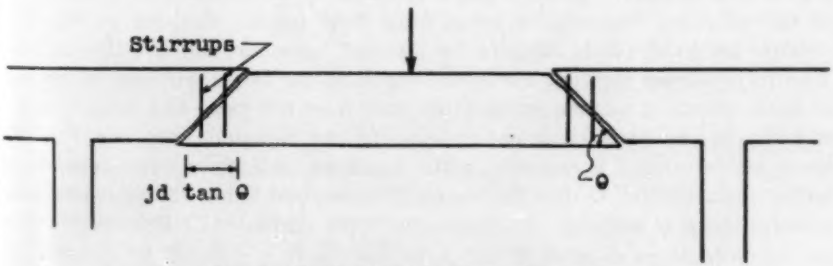

Fig. D—Cracks formed at inflection points

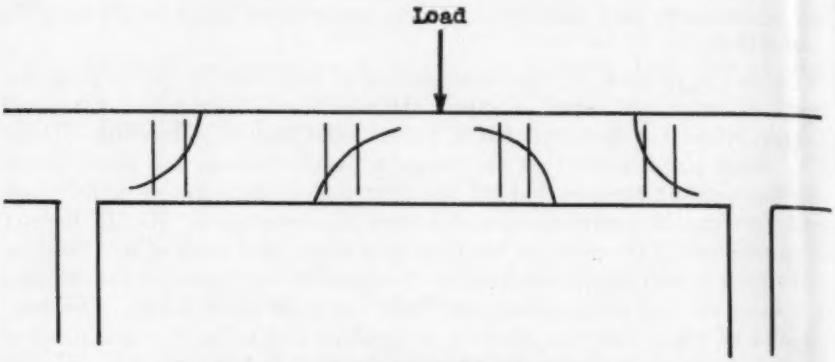

Fig. E—Cracks due to combined bending and shear

318-56 Section 800. When $\Theta = 45$ deg this is similar to Eq. (3), Section 803 of ACI 318-56 except that V, the total shear, is substituted for V'.

For inclined stirrups or bent-up bars, Fig. D leads to similar equations. For a single inclined bar or a group of inclined bars all lying in the same transverse plane, vertical equilibrium requires that $A_v f_v = v/\sin \alpha$. For several inclined bars or groups of bars, $A_v f_v \, jd/s = V \sin \Theta/(\sin \Theta \cos \alpha + \cos \Theta \sin \alpha)$. When $\Theta = 45$ deg these equations also, are similar to those of the ACI Code, with the total shear, V, substituted for V'.

From these equations, it would appear that, far from being *more* efficient than vertical stirrups, inclined shear reinforcement is actually *less* efficient. The reasons are that s is defined as the horizontal distance between bars, while the actual spacing becomes $s \sin \alpha$, and that the length of the bars required increases to $d/\sin \alpha$. Thus more steel is required to do the same job than when vertical stirrups are used. This is natural, since inclined bars cannot resist a vertical force as well as vertical bars can.

Fig. E shows possible modes of shear failure in the beam at points other than the inflection points. In each case, the crack must curve toward the horizontal in the compression zone. For the inverted keystone to drop out, either the crack must reach the compression face, which it cannot do in the region of compressive stress or else a crushing of the concrete in the direction of the principal compressive stress must take place. As long as sufficient stirrups are available to support the inverted keystone, the fact that a crack may have spread through the entire depth of the beam will not be critical. It must remain a hairline crack if the steel does not yield and bond is maintained. In the case where compression failure threatens, the fact that the shear reinforcement is supporting the keystone, will permit the concrete to adjust inelastically, so that the shearing stress, and hence the principal compressive stress is reduced. Furthermore, under current ACI Code restrictions on the percentage of steel or the f_c permitted, it is difficult to imagine the concrete becoming overstressed in compression even with the shear added.

To obtain some idea of the shear reinforcement necessary for the conditions of Fig. E, it is probably adequate to compute the reinforcement on the basis of an imaginary crack tangent to the curved cracks near the neutral axis. The location of the neutral axis to be chosen should probably be that at which the main reinforcement would just yield. In cases where the axial forces may be assumed to be small, a Θ of 45 deg would probably be adequate.

The necessary extension of the cracks in a more horizontal direction before failure can occur, seems to answer the problem raised by Mr. Rensaa in Fig. 3, where he stopped the cracks at the neutral axis and found that some of them were not crossed by stirrups. Furthermore, to obtain a compression failure of the type indicated in that figure, the beam would have to contain far more tension reinforcement than is permitted in the ACI Code.

It may seem that in this discussion the writer has been unduly pessimistic about the ability of the concrete to withstand some of the shear. However, it must be remembered that a few additional stirrups compared to the cost of a structure are cheap insurance against a type of failure that has been all too common in the recent past.

By K. HAJNAL-KÓNYI*

The paper is an excellent survey of our present knowledge on the controversial subject of shear, and its emphasis on "not to skimp on web reinforcement," thus approaching European practice more than the American, is sound. The principle that a beam, when loaded to its ultimate capacity, should not fail in shear, bond, or anchorage should form the basis of any future code.

The following two notes are intended to be complementary rather than controversial to the paper.

(1) It seems necessary to draw attention to the fact that the way the load is applied to a beam is relevant to the local distribution of stresses and to the capacity of the beam transmitting shear. Fig. F illustrates this point. In laboratory tests the load is generally applied as indicated in (a). In actual practice the most common case is (b) but cases like (c) do also occur. With-

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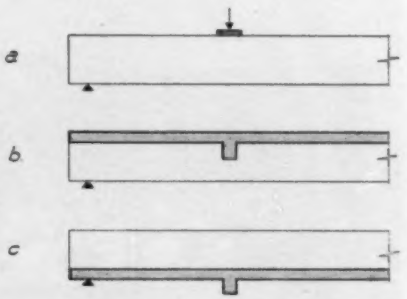

Fig. F—Three ways in which load is transmitted to a beam. (a) Laboratory test load arrangement. (b) Most common loading found in practice. (c) An alternate arrangement sometimes found in practice

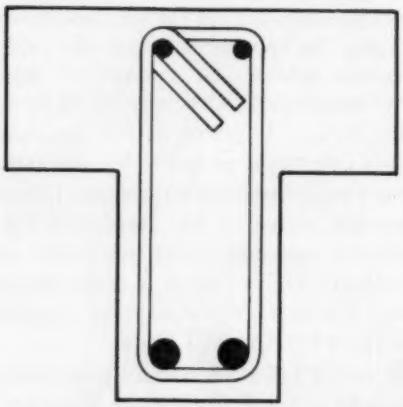

Fig. G—Arrangement of web reinforcement in test beams used by the writer[25]

out going into details it is obvious that case (c) is more unfavorable than (a) on which our knowledge and codes are mainly based. It would be desireable to carry out comparative tests to determine the reduction of ultimate load in shear by moving the position of the load from that shown in (a) to that shown in (c).*

Theoretical investigations of this problem on beams with great depth-span ratios are known for the non-cracked stage but not for beams of normal depth-span ratios at ultimate loads.

(2) The author says that "safe anchorage of the smooth stirrup bars in the compression zone was also a problem, although as a rule a lesser one."

The writer tested two series of T-beams a few years ago, each consisted of four beams, one series was reinforced with mild steel and the other an equivalent amount of Tentor steel.[25] In the first beam of both series the links were arranged as shown in Fig. G. While in the beam with mild steel the nominal yield stress of the reinforcement was reached at failure, the mode of failure (Fig. H) clearly indicated the weakness of the beam in shear. Within the shear span the web was separated from the flange by a horizontal crack which appeared before failure occurred. The ultimate load of the beam with Tentor steel that was not hooked was only 68 percent of the theoretical load. The appearance of this beam after failure (Fig. I) was similar to that of the beam with mild steel except for the bursting effect of the hooks. The crack in

*An indication of the reduction of the load bearing capacity of a beam in shear by moving the load downward may be seen from the tests by Ferguson, "Some Implications of Diagonal Tension Tests," ACI JOURNAL, V. 28, No. 2, Aug. 1956 (*Proceedings* V. 53), pp. 157-172.

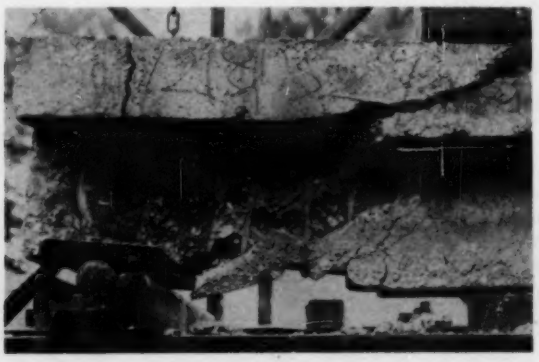

Fig. H—Beam of type shown in Fig. G, reinforced with mild steel, after failure

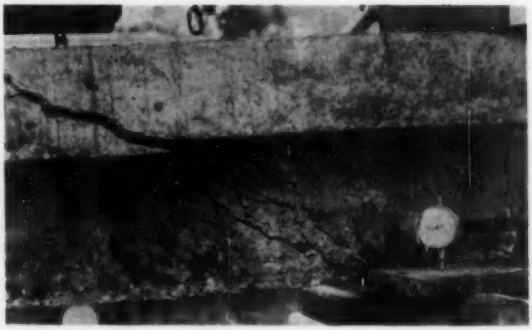

Fig. I—Beam of type shown in Fig. G, reinforced with Tentor steel, after failure

the flange over the support, clearly visible on the photograph, is indicative of a stress distribution as found by Watstein and Mathey.[26]

In the remaining three beams of both series the shape of the stirrups was changed, as shown in Fig. J, without increasing the total quantity of web reinforcement. Each of these beams failed in flexure by extensive elongation of the tensile reinforcement followed by crushing of the flange without any sign of weakness in shear.

The explanation seems to be the inadequate anchorage of the stirrups in the compression zone according to Fig. G although it complies with the requirements of the British Code of Practice CP 114 (1957) Clause 310g, which says:

Stirrups in beams and transverse ties in columns

Notwithstanding any of the provisions of this Code, in the case of stirrups and transverse ties, complete bond length and anchorage may be deemed to have been provided when the bar is bent through an angle of at least 90 deg round a bar of at least its own diameter; and the stirrup or tie is continued beyond the end of the curve for a length of at least eight diameters or, alternatively, through an angle of 180 deg with the stirrup or tie continued beyond the end of the curve for a length of at least four bar diameters.

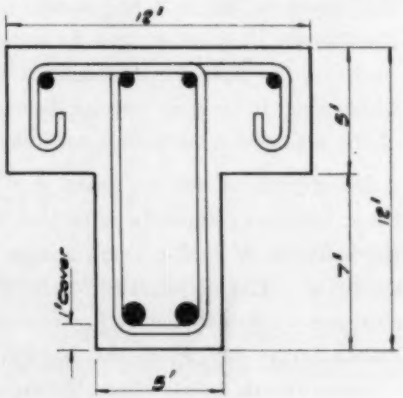

Fig. J—Alternated arrangement of web reinforcement in beams of tests conducted by writer[25]

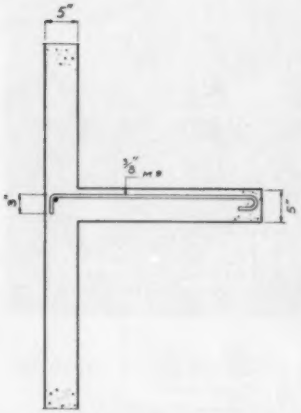

Fig. K—Cantilever slab reinforced according to British Code of Practice for stirrups

The requirements of this clause if applied to a cantilever slab would result in an arrangement shown in Fig. K and would obviously not be acceptable. Indeed, one cannot expect that the anchorage shown in Fig. K would enable the bar to develop its yield strength and there is no justification for allowing such an arrangement in stirrups in contrast to the requirements for anchorage of tensile reinforcement used otherwise. In this respect conditions in a T-beam are more severe than in a rectangular beam since the depth of the neutral axis smaller.

The writer should like to endorse particularly the author's comments on the web reinforcement of T-beams and on the need for stirrups to counteract the splitting effect of deformed bars at laps and bar endings.

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By MAX HERZOG*

The author is to be congratulated on his theoretical—not to be confounded with a mathematical—investigation of the problems mentioned in the title of his paper. The performance, and evaluation of the results of several hundred tests in the past has shown that no *generally valid* explanation of the phenomena in question can be expected from a purely empirical approach. Assisting the author's intentions I should like to stress shortly some points which seem to be clear beyond discussion, and to put forward a *new hypothesis* of the action of a reinforced concrete beam.

No objections will be raised to the statement that a reinforced concrete beam behaves elastically until the first crack forms. In this *uncracked state* the behavior of such a beam can be predicted with the help of the theory of elasticity. The trajectories of the principal stresses show with all necessary clearness where tension and compression are to be expected in the beam. As cracks form only when the tensile stresses surpass the tensile strength of

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concrete, we can fairly well predict where, and under which loads cracks will appear. According to the tests of Carneiro and Barcellos[27] with standard cylinders under static short-time load, the tensile strength of concrete can be expressed in terms of the compressive strength:

$$f_t = \tfrac{1}{3}\,(f_c)^{3/4},\ \text{kg per sq cm}$$

The well-known formula for shear stress in reinforced concrete beams of rectangular cross section,

$$v = \frac{V}{bjd}$$

can be found in the building codes of all countries and is, with the exception of a minor modification, identical with a formula for the theory of elasticity for the principal stresses in the height of the neutral axis of an ideally elastic beam. In the early days of reinforced concrete, $j = \frac{2}{3}$ as required by the theory of elasticity was substituted by the numerical value for j used in the section analysis, $j = \frac{7}{8}$ approximately. With the help of this formula we can, even today, obtain conclusive data on the occurrence of cracks in reinforced concrete beams, on the condition that we apply it to those beam sections only where its application is theoretically justified: the points of contraflexure, and simply supported beam ends. There only, the principal tensile stresses in the height of the neutral axis of the beam really are the largest tensile stresses of the entire beam section. In addition, proper credit must be given to direct tension or compression in the longitudinal direction of the beam, as already pointed out in the author's paper. The appearance of cracks in all other beam sections is governed by *bending stresses.*

A crack always runs at right angle to the trajectories of the principal tensile stresses, thus forming itself a trajectory of a principal compressive stress. Once a crack has formed, equilibrium of the *cracked section* can only be assured by suitable reinforcement crossing that section. Generally, three equations of equilibrium can be established for a cracked section of a reinforced concrete beam (Fig. L):

Moments about O:

$$M_a - T_s z - D u_D - S u_s = 0 \qquad (6)$$

Shear forces in a:

$$V_a - C_c \sin\beta - D \sin\alpha - S = 0 \qquad (7)$$

Direct forces in a:

$$N_a - T_s + C_c + C_c \cos\beta - D \cos\alpha = 0 \qquad (8)$$

As can be seen in Fig. L and in Eq. (7), the equilibrium of shear forces is essentially influenced by the angle β, between the compressive force and the horizontal.

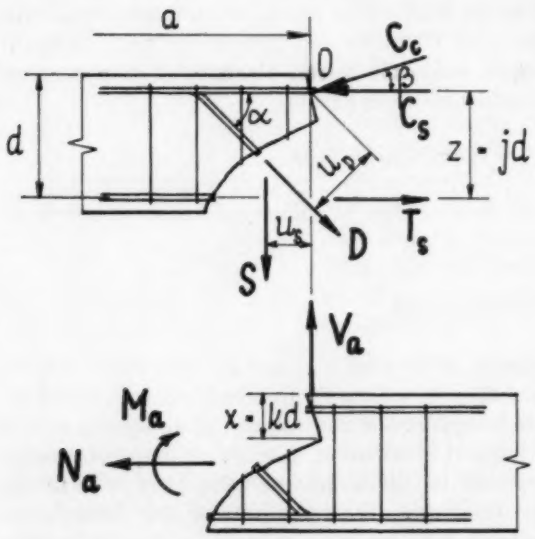


Fig. L—External and internal forces acting on the cracked section of a reinforced concrete beam

To get a better insight into the carrying mechanism of a reinforced concrete beam, we shall now consider two limit cases for a simply supported beam under uniformly distributed load. But just as in other theoretical speculations, here, we also cannot do without idealizing assumptions. Fig. M shows the first thought-model: a beam, the idealized, elastic carrying mechanism of which is represented by the two resultant trajectories of the principal tensile and compressive stresses. The tensile force diagram for the

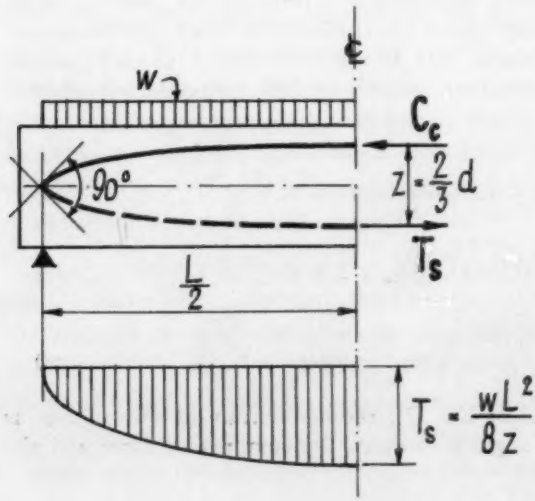
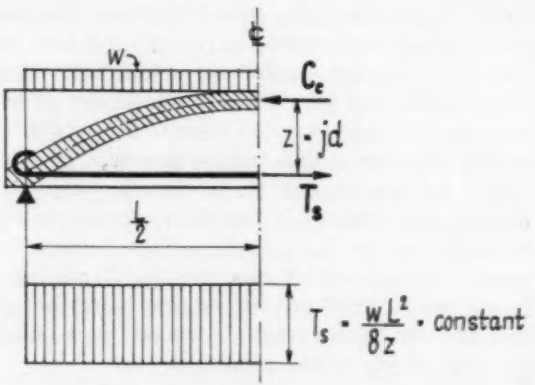


Fig. M—Idealized, elastic beam

Fig. N—Bow-string arch



curved longitudinal reinforcement, substituting the resultant trajectory of the principal tensile stresses, has the maximum value in the center of the span, and shows a reduction of the tensile force towards the support. The qualities of this idealized, elastic beam substantially depend on *ideal bond* characteristics. Fig. N shows the second thought-model: a beam, the idealized carrying mechanism of which is represented by a bow-string arch. The tensile force in the straight longitudinal reinforcement of this beam is constant over its whole span. *No bond at all* is required in the range between the supports, whereas *ideal anchorage* is assumed in the support regions of this beam. Comparing the two beams, we note one more essential difference. The compression center line of the first beam is almost elliptical, that of the second beam is a second degree parabola.

As ever, reality lies between the idealized extremes. Walther[28] has proved by experiments that full bond (i.e., no slip between reinforcement and surrounding concrete) cannot even be reached with bars with deformations, whereas the assumption "no bond at all" requires polished bars. Thus, the tensile force diagram and the compression center line of a reinforced concrete beam will lie somewhere between the first (Fig. M) and the second limit case (Fig. N); for beams with deformed bars it will be nearer to Case 1, for beams with plain bars it will be nearer to Case 2.

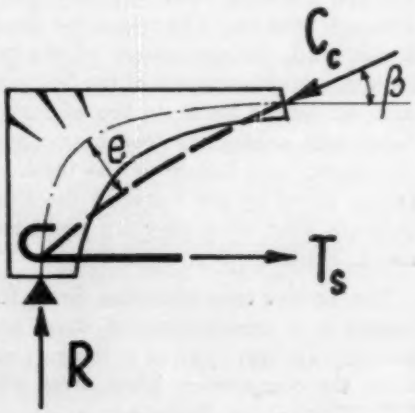


Fig. O—Portion of a cracked beam

Considering again a cracked reinforced concrete beam (Fig. O), we have to bear in mind two facts. First, cracks are trajectories of principal compressive stresses; in a simply supported beam under uniformly distri-

buted load their form is almost elliptical. Second, the compression center line of the actual beam will be intermediate in form between an ellipse and a parabola, as a consequence of the actual bond characteristics. Thus, concrete of the remaining part of the beam separated by the crack is acted on by a compressive force with the eccentricity e (Fig. O), producing tension on the top of the beam. This tension produces, in return, one or more cracks, and unless the compression center line lies inside the remaining concrete knee, collapse will follow. When the compression center line lies outside the remaining concrete knee, collapse can be prevented by additional reinforcement. We can either reinforce the remaining part of concrete as a knee frame, or, more effectively, arrange stirrups and/or diagonal bars. Taking into account the kinematic situation, it is quite obvious that diagonal bars are more effective than vertical stirrups.

The writer hopes, that this hypothesis of the carrying mechanism of a reinforced concrete beam might prove useful for the explanation of the carrying capacities of actual beams.

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By A. M. NEVILLE*

Mr. Rensaa's paper is valuable and stimulating for he tries to investigate the fundamentals of the problem of shear, and thus provoke us to think afresh.

The author is undoubtedly right in pointing out that the use of the nominal shearing stress as a basis of design is not correct. A constant working stress in shear has been found to result in a factor of safety varying from just over 1 to well over 10. The reason for this is that, once the diagonal tension crack has opened, the conception of the nominal shearing stress becomes meaningless, and the strength of the beam depends on the resistance of the section *modified* by the crack, to the compressive and tensile forces. This has been taken into account in the shear-compression formulas of Laupa, Siess, and Newmark,[7] and failure in the tension zone has been observed in numerous beams tested by the writer at the University of Manchester. No numerical data are given since they are presented in a paper just written by the writer and J. Taub.

The writer's tests have also shown that the provision of a flange in T-beams results in a modification of shear behavior as compared with rectangular beams. The presence of a flange precludes a shear-compression failure because the compression block, even when reduced in depth by the diagonal

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tension crack, is large enough to prevent crushing. Failure can therefore occur either by the yield of the tension steel due to flexure or by the disruption of the tension zone between the bottom of the diagonal tension crack and the support. The exact manner of this failure depends on the type, layout, and anchorage of the reinforcement.

A comparison of rectangular and T-beams of the same width (i.e., with a web width of the T-beam equal to the width of the rectangular beam) and with the same reinforcement, has shown that T-beams can take a load of up to about 60 percent greater than rectangular beams, both types of beams failing in shear. This increase in the load-carrying capacity depends on the a/d ratio. It seems then that Mr. Rensaa is not justified in his criticism of the ACI Code regulations concerning the shear reinforcement in T-beams.

The author is right in drawing our attention to the details of the dowel action but perhaps rash in disregarding its contribution to shear resistance. However, the contribution of the various elements—main steel, stirrups, and concrete—is of little interest. It is essential that the widening of the diagonal tension crack and the splitting of the beam be prevented and the integrity of the beam be preserved. Thus the author correctly says that "stirrups will perform the useful additional function of reinforcing a beam transversely against splitting," although the writer doubts whether this splitting is due to bond stresses rather than to the effects of shear.

Likewise, the author's observations on the distribution of stresses in the tension steel are of interest, but here again, not bond, but excessive diagonal tension cracking is believed to be the primary cause of failure.

The paper is an interesting study and it is hoped that it may lead to some more detailed design recommendations.

By R. B. L. SMITH*

The introduction and aims of Mr. Rensaa's paper are admirable but unfortunately the development does little to advance the study of shear in reinforced concrete beams and cannot be said to have caught up with recent research. In spite of his plea for a re-examination of fundamental ideas, he does not ask the questions: (a) does shear directly cause the failures of reinforced concrete beams? (b) how do its effects contribute towards such failures?

The answers to these questions are now largely known, and they reveal that many of the problems Mr. Rensaa tries to solve in vain, as many others have done over the past 50 or so years, do not exist. For example: The first line of the last paragraph on p. 701—it is more than "not easy" to determine the actual distribution of shear load between concrete and web reinforcement. It is impossible, since the problem is itself invalid. In the same way it is futile to ask on p. 698, "which contention is right, the European or the North American?" when the answer is, neither.

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Another example of an outworn question, which has been fruitlessly asked many times, is implied in the top paragraph on p. 700. Web reinforcement does not "carry shear," it reinforces a beam to resist the effects and restrict the development of diagonal cracking.

It is surprising that he can on the same page refer to the work of Laupa, Siess, and Newmark,[7] accusing them of trying to work out a formula for the ultimate shear strength of beams. The chief point of their paper referred to by Mr. Rensaa, which is a landmark in the research on this subject, is that shear must be considered in terms of its effect on the resistance moment of a beam. Furthermore, if beams tested in laboratories are not to be accepted as evidence, where are we to base our theories? We can hardly hope for sufficient failures of actual structures under sufficiently well-controlled conditions to advance the subject. It is easy to criticize the limitations of laboratory experiments, but the knowledge thus gained affords a series of small steps away from the present day attitude of ignorance.

Considering the stress trajectories shown in Fig. 1, Mr. Rensaa says: "The average slope of a stress trajectory is steeper than 45 deg when near the support and becomes almost vertical near the center of a beam." Perhaps there is some error here. On p. 703 he goes on to ask, "If the crack is so steep that stirrups will not necessarily cross it, of how much value are the stirrups?". The answer is that if the cracks are vertical due to horizontal stress trajectories of Fig. 1, they are flexural cracks and stirrups are not required. On the other hand, if they occur in beams with concentrated loads close to the supports, they are probably not tension cracks at all, but proper shear cracks. In which case stirrups will only serve the purpose of holding the reinforcement together. It may be added that this type of failure is only possible when concentrated loads are within a distance from the support less than the beam depth, that anchorage failure is likely to precede it, and if the shear span is any greater, "shear-compression" (moment) failure will intervene to preclude it.

On the same page, the author advocates the use of sloping stirrups. These are not useful in practice since they do not form a rigid cage of reinforcement as do vertical stirrups. They are of course excellent in theory.

Mr. Rensaa's discussion of shear conditions at points of contraflexure is unconvincing. He goes to some length to prove the elastic shear stress theory inapplicable, but surely it is as valid here as anywhere else? The elastic theory merely assumes that concrete is incapable of resisting tension due to bending and this assumption may be made at any section. It is submitted that sudden failures of beams at the assumed points of contraflexure are due to:

(a) Initial diagonal cracking developing in the absence of web reinforcement, leading to splitting along the main reinforcement accompanied by loss of bond.

(b) Insufficient anchorage (i.e., grip length) of the main steel either side of the assumed point of contraflexure which combined with (a) leads to disastrous failure.

Axial tension due to shrinkage may lower the incidence load for diagonal cracking, but does not affect the essential mechanism described. If this is prevented, then there is no reason to consider a point of contraflexure as an especially weak region as argued on pp. 708-709.

On p. 714, Lines 4-7 have a strong flavor of the famous "simple rules" in *Alice in Wonderland*, and in Lines 13 to 15, does the author argue that every beam requires compression reinforcement or merely nominal or designed stirrups anchored to top tie rods?

AUTHOR'S CLOSURE

The discussers have brought up several questions which would warrant extensive discussion but this will be limited to only a few of the points mentioned.

Professor Gesund is of the opinion that vertical stirrups may be more efficient in carrying vertical shear than bent up bars. Both his sketches and Fig. 2b could be interpreted as proof of his contention. The writer might also have agreed, were it not for the result of comparative tests which prove otherwise. In this respect reference is made to the extensive German tests[8] and also to tests made in the United States (Reference 2, chapter 4). These seem to show that bent-up bars are somewhat more efficient than stirrups but that a combination of stirrups and bent-up bars with the latter taking the major part of the shear is the best system of shear reinforcing. If we accept this, it will then be necessary to start from this point and try to find out what makes bent-up bars more efficient. Several writers have pointed out that stirrups do not act with much efficiency before a crack has opened up to some extent. That is also quite understandable because some transverse bending of the legs of a stirrup must take place before it becomes effective and can take longitudinal tension. They act afterwards partly as dowels and partly in direct tension. A bent-up bar, which generally crosses a crack more at a right angle, will get into action at once when a crack starts to form and does not require any transverse bending to do so.[29] It is even possible that in addition bent up bars may have a dowel effect in some cases. Bent up bars would of course be much more effective as dowels than stirrups because of their generally larger cross section. Such bars will therefore have a much greater value in preventing a crack from increasing and this is a rather important factor. Furthermore, bent-up bars are not so likely to fail progressively as stirrups are and they have generally a better anchorage than stirrups have in some cases when the crack comes near the stirrup ends. Bent-up bars are for the above reasons more efficient and reliable, if crushing at the bends is guarded against. In addition it should be recognized that stirrups will only take the vertical component of the diagonal tensile stresses and that more longitudinal reinforcing and better anchorage will be needed to make up for the deficiency of horizontal web reinforcing.

The discussion by Dr. Hajnal-Kónyi is a valuable contribution to the problem of stirrup anchorage. His tests certainly prove how important proper

anchorage of stirrup ends is even when the stirrups are made of deformed bars. This information should influence future anchorage requirements. It is not unusual to see stirrups anchored with ordinary hooks on the tension side on the portion of a beam acted on by a negative bending moment. The writer has often wondered how effective such stirrups are. It was not, however, the writer's intention to minimize the importance of stirrup anchorage. Proper anchorage is, of course, of special importance when stirrups are used for carrying the main part of the shear.

Dr. Herzog presents an interesting theoretical discussion of stress conditions at and near a support. The exact stress distribution at the support of a beam is complicated. A solution might possibly be obtained by means of the mathematical theory of elasticity, although bond slip would complicate its use. Photo-elastic stress analysis with a model simulating the properties of reinforced concrete could perhaps also be used. It seems possible that the stresses on the portion of a beam on a distance equal to about the depth of the beam would be much like those for beams of high depth-span ratio.

The writer wants to correct the expression on p. 709 in the second paragraph, under Flange Effect, that "at a freely supported end with ordinary proportions of flange and stem, the flange will have a negligible effect on preventing initial diagonal tension cracks in the stem." A correct statement would be as follows: At the freely supported end of a beam the strength of a beam to resist initial cracking will depend on the tensile strength of the concrete and on the section modulus and moment of inertia of the uncracked concrete section. These properties will govern the initial strength in bending and shear of the section. The presence or not of reinforcement will have little or no effect. Since a T-beam will generally have a larger section modulus and moment of inertia than a rectangular beam of the same height as the T-beam, and with the same width as the stem of the T-beam, we should then expect a correspondingly greater initial cracking load.

In answer to Dr. Neville's discussion, it is not difficult to understand that a T-beam with equal height to a rectangular beam and with a web width equal to the width of the rectangular beam could be up to 60 percent stronger. It is to be expected that on the portion of a beam where both shear and bending moment are a maximum and the flange is on the compression side, the T-beam would show a greater ultimate strength. The comparison would not be fair, however, since the lever arm of internal stresses would not be the same for the two cases and that was a necessary condition as stated in the paper. The flange, if on the compression side, will undoubtedly increase the shear-compression strength of the beam. If the initial cracking load was followed by an immediate failure in both cases, it would then be justified to say that T-beams are much stronger in shear than rectangular beams. That would apply even if the flange was on the tension side of the beam and provided there was no appreciable axial tension from shrinkage restraint in the flange. Such failures would be of the brittle kind which we do not

want. We should, therefore, require that the section of concrete remaining uncracked, together with the reinforcing crossing a crack, have a greater strength than the original concrete section. If there is insufficient strength in the reinforcing to make up for the loss of the cracked portion, such reinforcing would be of no help and failure would follow immediately after the initial crack had formed. If the residual section of concrete together with the reinforcing is stronger than the original concrete section, there would then be no immediate failure but as the load increased, the crack would extend farther up into the remaining concrete section and make it less and less effective. The area of concrete contributing to the carrying of shear is, therefore, not constant but will decrease as the load increases. In the case of the flange being on the tension side of the beam a crack would pass through the flange long before failure making it completely ineffective to resist any shear.

Near the end of a freely supported beam where the bending moment is small but the shear is a maximum, the flange would lose most of the advantage it gives where both shear and bending are high excepting perhaps some of its influence in lengthening the internal lever arm. At such a point, failure would be mostly due to diagonal tension as the compressive strength of the flange would not be of much importance.

The writer's criticism of the 1956 ACI Building Code was clearly stated to refer to Section 801(e). This clause specifies the use of web reinforcing on a beam from the support to beyond the extreme position of the point of contraflexure, that is, on the portion of a beam acted on by a negative bending moment. The flange on this portion is in practically all cases on the tension side and how the presence of such a flange can preclude or hinder a shear failure is beyond the writer's understanding. If it is wanted to lower the requirements for web reinforcing in a T-beam, it should then be clearly stated under what condition this can be done safely. Making a general rule out of the most favorable condition is the same as setting traps for the unwary. The writer's criticism of this clause in the ACI Code seems therefore, well justified.

It is not unusual to see cracks caused by concrete shrinkage across slabs which are long and heavily restrained against contraction even if there is no bending of the slab at such a point. The force required to form such a crack which often occurs after the concrete has obtained almost its full strength, is quite large. Assume for instance a concrete tensile strength of 300 psi which is not unusual for good concrete and a 4 in. thick slab. The force required per linear foot to tear this slab apart will then be $300 \times 4 \times 12 = 14{,}400$ lb and for an 8 ft wide slab, for instance, 115,200 lb or 57.6 tons. That is certainly not something negligible if we want to make exact calculations. On the portion of a T-beam acted on by a negative bending moment, the tension from bending and that from shrinkage, whatever magnitude this is, will be additive, while on the portion of the beam with positive bending moment they will be subtractive. A crack in the top flange is, therefore, most likely to develop first near the interior support but if the beam is long a second

crack may also occur in another place nearer the point of contraflexure. When the slab flange cracks, the shrinkage reinforcing will come into action and will carry some of the tension previously held by the slab. However, if only the minimum code requirement has been followed for the shrinkage reinforcing, this will generally be insufficient to carry all the previous tension in the concrete. The shrinkage reinforcing will be stressed beyond its yield point and remain in tension corresponding to this. The balance of the total tensile slab stress lost will then have to be transferred to the adjacent beams and carried by their reinforcing, which fortunately is sufficient also for that purpose in most cases. It is quite usual to find that a visible crack, in for instance a long corridor slab, will diminish towards the longitudinal corridor beams and get out of sight there. However, it is quite certain that the extra shrinkage tension does not disappear at the beam but has to be taken up by this in addition to its usual bending stress. It seems under these circumstances even more unrealistic to believe that a flange on the portion of a beam acted on by a negative bending moment has any value in resisting shear. It would also appear that some extra shrinkage reinforcing at the top of a slab adjacent to a beam would be beneficial on the negative moment portion.

The writer thinks that it is a proved fact that longitudinal tension in a bar embedded in concrete in addition to bond stresses also creates transverse tensile stresses in the surrounding concrete. This concrete then is stressed in tension, first from shrinkage, secondly from a transverse component of anchorage stresses and, thirdly in some cases from dowel action. The combination of the first two stresses mentioned generally does not leave much strength for the third in the case of the main reinforcing. Dowel failure is also apt to occur before the ultimate load is reached which makes it even more reasonable to neglect its effect on shear strength.

Professor Chinn's information that compressive stresses in reinforcing have been measured and found to be of about the same magnitude as calculated by the writer is interesting. These stresses will, of course, vary with the steel percentage and the concrete properties.

The diagonal cracks in the beams shown near the support in Fig. A could probably have been avoided if top bars of a reasonable size had been used as the writer has recommended. There could at the upper end of the crack be a small negative bending moment caused by the column restraint and no reinforcing for such is shown. In the case of the unbroken slab this would have some tensile strength which most likely prevented formation of diagonal cracks. It may also be mentioned that in a case like the one shown with the lower end of crack close to the support, there will be a considerable safe dowel action which will be a help in this case.

Mr. Falconer presents a theoretical discussion which is not easy to follow in all its details. The writer is, however, pleased to note a general agreement in the majority of cases.

Regarding points of contraflexure the writer based his treatment on the simple and safe assumption that if a crack first starts at such a point it would

then continue to go through the beam. There is, of course, a possibility that a bent-up bar passing through the middle of a crack under formation could stop it from further extension, but it does not seem safe to figure with such a possibility. If a crack has severed a beam completely, any theory that includes action of concrete along or across such a crack must necessarily be erroneous.

The writer would be in full agreement with the inclusion in a code of Mr. Falconer's Eq. 3(a) which is identical to Rausch's formula, whose deduction was shown in the paper. Whether Eq. 4(a) is preferable to the ordinary simple rules is more difficult to say.

In general, it seems doubtful that we shall ever be in a position to figure stirrup requirements exactly. There are too many uncertainties including the strength of concrete in tension and above all, the uncertain and variable effect of shrinkage and support restraint against longitudinal movement. These effects are to some extent of a secondary order as far as ordinary bending is concerned but are of primary importance when it comes to shear and diagonal tension resistance. The situation would look rather hopeless were it not for the fact that we know for certain that the maximum web reinforcing required will be for the full shear. How well does it then pay to save a little web steel by time absorbing calculations based on uncertain basic assumptions? If we make wrong assumptions. the result may be both failure of the structure and ruin of the designers reputation. The writer would answer this question with a quotation from Professor Gesund's discussion:

"It must be remembered that a few additional stirrups compared to the cost of a structure are cheap insurance against a type of failure that has been all too common in the recent past."

The writer would like to express his gratitude to those who have taken part in this discussion and contributed to the accumulation and extension of knowledge concerning the difficult and contentious questions considered.

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Discussion of a paper by D. Watstein and R. G. Mathey:

Strains in Beams Having Diagonal Cracks*

By MICHAEL CHI, R. B. L. SMITH, J. TAUB and A. M. NEVILLE, and AUTHORS

By MICHAEL CHI†

The authors reported some interesting but unexpected results. In short shear spans, strains are known to be nonlinear to a moderate degree.[1] The extreme departure from linear distribution of strains and considerable shifting downward of the neutral plane cannot be completely explained by influences of shearing strains and local compression by point loads. The following analysis was developed in an attempt to understand and explain the experimental results.

Reinforced concrete is equivalent to a homogeneous material by virtue of transformed section. Like all homogeneous materials, a reinforced concrete beam owes its beam action to adequate resistance to horizontal shear—specifically, the strength of bond and rigidity of strips of concrete between cracks. When this resistance is adequate, resultant compressive stress in a rectangular beam remains a distance of 1/3 *kd* from the top extreme fibers, shown in Fig. A as line *ab*.

As soon as this resistance is overcome by shear, the member ceases to behave like a true beam and acts more like a tied arch. This was pointed out by Komendant[2] in his Fig. 2. Moment in the beam remains the same as predicted by laws of statics. Tensions in steel, however, are greater than those developed by beam action. The accompanying increase in the resultant of compressive stresses in concrete must bring its point of application much closer to the steel level, as illustrated by line *acd* in Fig. A.

Pressure line *acd* has the following effects. First of all, the distribution of strain can no longer be linear. Only a big bulge of stresses near the mid-depth and a down shift of neutral axis, as shown in the left diagram of the authors' Fig. 4, can produce resultants indicated by line *acd*. Extreme fiber stress must also decrease—even changing to tension. Secondly, the rapid extension of steel promotes prolongation of cracks. Because of the presence of curved pressure line *acd*, cracks are deflected to flatter slopes, following curved paths, and converging to the loads; the one nearest the support becomes the so-called diagonal crack. Thirdly, the vertical component of the

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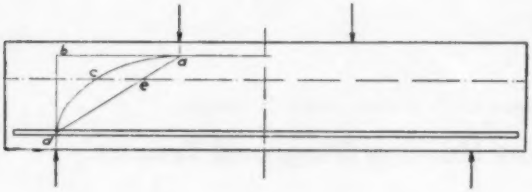

Fig. A—Pressure lines in shear span for different degrees of resistance to horizontal shear

resultant along line *acd* is usually smaller than the external shear. The unbalanced shear must be taken by the steel. The short lengths of steel exposed by cracks are usually capable of providing this dowel action.

When the ability of the beam to resist horizontal shear is completely destroyed, resultants of compressive stresses in concrete follow a straight path as shown by line *aed* in Fig. A, while the tensions in steel at support and at midspan are equal. It is obvious that the vertical component of line *aed* is equal to the external shear and the steel need not take any shear. Therefore, the shear taken by steel reaches a peak value just when beam action is broken, and gradually drops to zero when destruction of shear resistance is completed, as indicated in the author's Fig. 8. It is noteworthy that resistance to horizontal shear depends on the combination of strength of bond and rigidity of strips of concrete between cracks. Closely spaced cracks would destroy the beam's resistance to horizontal shear regardless of the strength of bond.

It is seen that the diminishing ability of the beam to resist horizontal shear can be considered as the cause of its mysterious behavior. Formation of the diagonal crack may be a consequence rather than a cause of it.

Incidentally, strain gages on the concrete surface measured concrete strains but could not detect displacement of one gage relative to the other. Strain data at lower loads, reported by the authors, indicated that concrete strains were proportional to their distances from the neutral axis but this did not prove that plane sections remained plane. In other words, strains in a shear span obeyed Navier's law at low loads but not Bernoulli's hypothesis.[3] Invalidation of the latter in a shear span has been amply exposed by standard texts.[4] This minor error surely should not mar the authors' excellent presentation of interesting test results.

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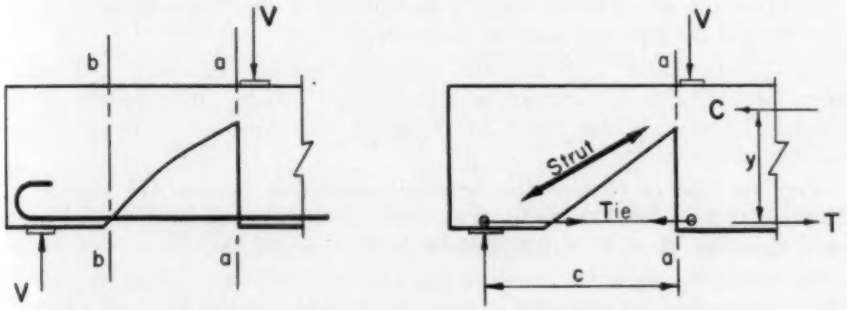
By R. B. L. SMITH*

The free body analysis of shear transference is of great interest since it is used in conjunction with the experimental evidence of steel strain measurements. It is reported that the maximum value of V_1 occurs when the diagonal crack first approaches the section at the applied load, i.e., immediately before bond breakdown along the steel between sections a and b in Fig. 7. This indicates that the greatest danger of horizontal splitting between the main steel and the body of concrete above it in the region between section b and the support occurs at this stage of maximum dowel action. In short shear spans such as those of the author's tests, such splitting is prevented by the fact that the bottom of the diagonal crack almost runs into the support, but with longer shear spans this is a most common form of failure of beams without stirrups, usually reported as diagonal tension failure, since it occurs as soon as the diagonal crack is fully developed. The horizontal splitting may lead to anchorage failure or (since bond failure between sections a and b is imminent, leading to constant tension in the steel from section a to the anchorage on a small increase of load) the steel strain increases rapidly causing excessive rotation and failure about the top of the diagonal crack.

If this horizontal splitting is prevented, by using stirrups in the region between section b and the support, the bond failure between sections a and b, as the load is increased, results in a condition equivalent to:



for which $Vc = Ty$
= bending moment at section a

Thus using the authors' analysis, $V_1 = 0$, i.e., all the shear force must be carried by the uncracked section at the top of the diagonal crack. In spite of this, however, it is observed in tests that this section does not shear through. As the load increases, it either fails in flexural compression as it reduces in area, as described by Laupa and others, or else the diagonal crack continues as an almost horizontal split following a direction perpendicular to the principal tension. This latter development may be restricted by the use of stirrups,

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which therefore may be of value outside the shear span, in a function complementary to that of preventing splitting along the main steel referred to above.

Stirrups in the region between sections a and b influence the mechanism whereby diagonal crack penetration of the compression zone leads to flexural failure. Since they act in tension, if these are superimposed on the free body diagram, it will be seen that the direction of the force V_1 (i.e., the dowel action) must be reversed after bond failure.

The right hand sketch above incidentally explains the strain reversals in the top surface of the concrete observed by the authors, since the "strut" is eccentrically loaded.

By J. TAUB* and A. M. NEVILLE†

Messrs. Watstein and Mathey are to be commended on a clear investigation and presentation of test results. The authors' report was especially interesting to the writers as they have just prepared a paper on the resistance of reinforced concrete beams to shear.

On the basis of data presented by the authors the following picture of beam behavior seems to be probable.

DISTRIBUTION OF COMPRESSIVE STRESS IN CONCRETE WITHIN THE SHEAR SPAN

The strain distribution for beam A-15 is given in Fig. 4. On the basis of this figure the stress distribution in Section III of this beam under loads of 15, 35, and 55 kips will now be considered.

(a) Up to load $P = 15$ kips the stress distribution was linear. No cracks were formed in the shear span, and the neutral axis lay at middepth of the beam. Up to this load the beam behaved, therefore, like a beam of unreinforced concrete.

For the load of 15 kips the bending moment at Section III was $M = 7500 \times 18.5 = 138{,}750$ in.-lb. The stress at the extreme fiber of the beam was, therefore, $f_c = (6 \times 138{,}750)/(6 \times 15^2) = 617$ psi.

According to Fig. 4 the strain at the extreme fiber was 200×10^{-6}, hence the average value of modulus of elasticity of concrete was 3×10^6 psi.

(b) Under the load $P = 35$ kips two major diagonal cracks in the shear span were well developed. In Fig. B(a) the points of intersection of these cracks and Section III are marked by M and N, respectively. As it can be seen from Fig. 4 the point M coincides with the level of the neutral axis. The low position of neutral axis is to be noted; its distance from top of the beam was about 9.8 in.

On the basis of strain measurements at Section III the curve of compressive stress in concrete was traced in Fig. B(b). This curve was drawn according to the following assumptions:

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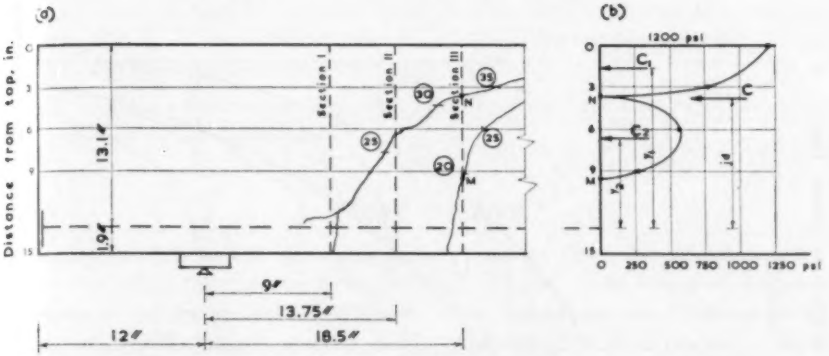

Fig. B—Stress distribution in concrete at Section III for $P = 35$ kips

1. Since no compressive stress can be transferred across the diagonal crack the internal compressive force at points M and N is equal to zero.
2. The average value of modulus of elasticity of concrete is 3×10^6 psi.

According to Fig. 4 the maximum strain for $P = 35$ kips occurred at the extreme fiber of the beam and amounted to 400×10^{-6}. The maximum compressive stress there was, therefore, 1200 psi.

The diagram of compressive stress in concrete, shown in Fig. B(b), consists of two branches. This seems to explain the low position of the neutral axis. The upper component of internal compressive force is $C_1 = 20{,}600$ lb; its distance from centroid of tension reinforcement, $y_1 = 11.5$ in. The corresponding figures for the lower component are: $C_2 = 15{,}300$ lb, $y_2 = 6.4$ in. The resultant compression force in concrete yields to 35,900 lb; its distance from centroid of tension reinforcement is 9.3 in.

The correctness of the above figures will be checked as follows. The bending moment at Section III under the load of 35 kips is $M = 17{,}500 \times 18.5 = 323{,}750$ in.-lb. Hence the tension force in the longitudinal reinforcement, $T = M/jd = 323{,}750/9.3 = 34{,}800$ lb. It is seen that this value is in fairly good conformity with the above calculated value of the compression force C.

(c) Up to load $P = 35$ kips the maximum compressive strain in concrete was developed at the extreme fiber of the beam. For higher loads the maximum compressive strain in concrete occurred some distance below the extreme fiber. For $P = 55$ kips this distance was 3 in. at Section III and about 6 in. at Section I. For loads exceeding 45 kips, even a small tension strain occurred at compressive face of the beam at Section I.

This seems to indicate that a tied arch action was now developed in the beam as shown in Fig. C. The strain measurements correspond well to this assumption.

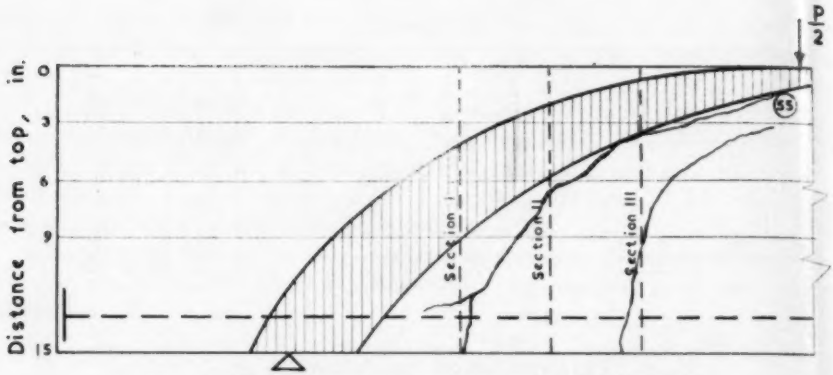

Fig. C—Tied arch action developed under higher loads

STRESS IN THE REINFORCEMENT WITHIN THE SHEAR SPAN

In six beams, strains in steel at a section 3 in. from the support and at midspan were measured. Up to the load where no cracks in the shear span appeared, the tensile stress below the neutral axis of the beam was taken by the concrete and the stress in the reinforcement was practically equal to zero. After formation of the diagonal crack the stress in tension steel at intersection with this crack increased rapidly. With increasing load the crack extended toward the load point and opened wider. At intersection with the tension steel it pressed down this steel in the direction of its opening; the force in the longitudinal reinforcement acted, therefore, inclined. Fig. D shows the beam deformation for this stage of loading.

From this figure it is seen that the tension steel now transfers a significant part of the vertical shear. The measurements by the authors showed that at loads ranging from 0.42 to 0.46 of the maximum the ratio of the maximum transferred shear to the total shear in the beams ranged from 0.34 to 0.74.

With further increase of loading, the force in the tension steel at intersection with the diagonal crack greatly increased, and the bond between this steel and the concrete was successively destroyed on both sides of the crack. Under action of this force the longitudinal steel at the crack crossing became straightened and its inclination toward the beam axis decreased. According to the conditions of equilibrium the three forces, C, T, and V, must meet at one point; the inclination of the compressive force in concrete,

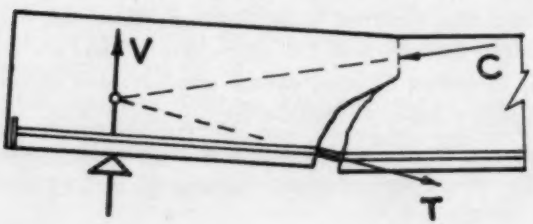

Fig. D—Beam deformation after the diagonal crack was fully developed

therefore, now increased. A tied arch action in the beam was developed as shown in Fig. C. The shear transferred by the concrete now became greater and that by the tension steel accordingly smaller.

The strain measurements by the authors seem to confirm the above description of beam behavior.

AUTHORS' CLOSURE

Mr. Smith's discussion advances the hypothesis that the formation of a diagonal crack is followed by "breakdown" of bond along the steel between sections *a-a* and *b-b* in Fig. 7. While it is true that the bond stress decreased sharply along this length of steel on the development of a diagonal crack, this occurs by reason of the fact that the tensile stress increases sharply at section *b-b* and not because of "breakdown" of bond which we normally associate with excessive slip of the steel bars.

According to the authors' observations, the portion of the shear transferred by dowel action of the reinforcement decreases with an increasing load, and as the cracks widen the fractured segments of concrete lose their ability to transfer vertical shear across the cracks. This view of the mechanism of shear transfer by dowel action and the fractured segments of the concrete beam is confirmed by the data given in Fig. 8. It can be seen that the specimen B-18-2 which contained 3.05 percent reinforcement stressed to 33,000 psi at failure still transferred about 25 percent of the total vertical shear at a load equal to 0.86 of the maximum. Specimens D-18-1, D-18-2, and E-18-1, with smaller ratios of reinforcement (0.75 to 1.16 percent) which had steel stresses of 70,000 to 88,000 psi at failure showed little ability to transfer vertical shear as the load approached the maximum.

Mr. Chi stated that the paper presented ".....unexpected results." The authors' observation of nonlinear strain distribution within the shear span and even the presence of tensile strain at the extreme fiber of the compressive zone was not entirely unexpected. Similar results attending the redistribution of stress following formation of diagonal cracks were reported by Morrow and Viest[5] and Hanson.[6] Hanson also advanced a hypothesis which satisfactorily explained the nonlinear strain distribution in the concrete observed by the authors.

Mr. Chi in his discussion correctly attributes the diminishing ability of the beam to transfer vertical shear by dowel action as the load increases to the gradual destruction of the shear resistance of the cracked sections of the beam. It does not appear, however, that the resistance to shear is dependent on the rigidity of the segments of the beam between adjacent cracks, or the spacing of cracks. We may refer again to Fig. 8 and point out that the beam with the largest percentage of reinforcement (B-18-2) which had the same crack spacing as the beams with lower ratio of reinforcement, proved to be most durable with respect to transfer of shear by dowel action. It is

concluded that it is the width of cracks in the shear span that determines the ability of the cracked beam to resist shear.

Mr. Chi correctly pointed out that the shearing stresses cause a plane section to become warped, so that strictly speaking we cannot refer to a plane section remaining plane within the shear span. The authors tacitly neglected the effect of the shearing strain in producing distortion of a plane section to bring out more clearly the departure of the flexural strain distribution from a linear one upon formation of a diagonal crack.

The authors appreciate the discussion of Drs. Taub and Neville which presents a clear analysis of the behavior of the beams following formation of diagonal cracks.

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Discussion of a paper by K. C. Ray:

Influence Lines for Pressure Distribution Under a Finite Beam on Elastic Foundation*

By H. S. GEDIZLI, GERALD PICKETT, DRONNADULA V. REDDY, ANDREW RETI, DONALD A. SAWYER, and AUTHOR

By H. S. GEDIZLI†

We know that the first differential equation of the paper is only true in the case of a grade beam which is imbedded in a material capable of exerting downward as well as upward forces on it and which is also infinitely long. Otherwise, there are breaks in the deflected earth surface, corresponding to the ends of the beam; this contradicts the principle of continuity.[1] Its use is not advisable for finite grade beams. In the writer's opinion, it would be much more suitable to follow Pozzati's method of analysis of finite grade beams,[2] which is simple, clear and exact.

By applying this method to the same example as in the paper, it can be easily shown that the final results, i.e., the moments, shears, and pressures of the finite grade beam, are obtained by superposition of the moments, shears,

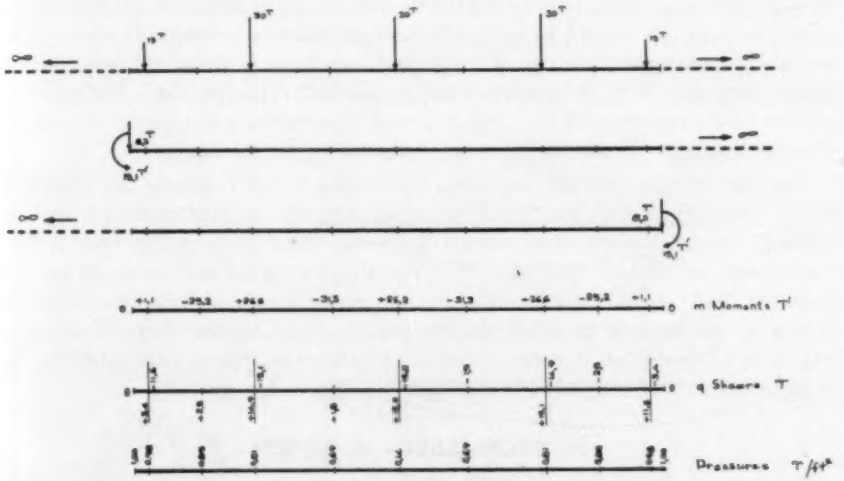

Fig. A

*ACI Journal, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), p. 729. Disc. 55-47 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 12, June 1959 (*Proceedings* V. 55).

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and pressures of three infinite beams, namely; *one* infinitely long grade beam bent by the same concentrated loads; *one* rightly semi-infinite beam bent by a moment and a concentrated load at left end as shown on Fig. A; and *one* leftly semi-infinite beam bent by a moment and a concentrated load on right end as shown on Fig. A. The numerical results are also recorded on Fig. A. The discrepancies between the moments as well as shears are not important but the discrepancies between the pressures are considerable. It is important to remember that these two methods of analysis of finite grade beam are only applicable in the case that all pressures are positive, i.e., they are upward forces. If however some of them are downward forces we must follow Schindler's method of analysis.[3]

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By GERALD PICKETT*

The author is to be commended for converting the theoretical solution of this problem to a form readily usable by the practical designer. There is one point, however, in regard to subgrade modulus that is not entirely clear. The author indicates that the use of a subgrade modulus is valid but does not indicate how one is to determine what numerical value to use. Perhaps the author would recommend that which would be used for a pavement slab under interior loading. If this is the case it should be so stated.

The use of the method of finite differences usually results in relatively simple derivations but may require rather lengthy computations for desired accuracy as the author discovered. However, this handicap has now largely been overcome with the increase in availability of digital computers of greater capacity and speed. Those with these more modern facilities available should be encouraged to make similar contributions by the author's example which was accomplished without benefit of these machines, but with the aid of assistants using only desk calculators.

By DRONNADULA V. REDDY†

The writer has read with interest Mr. Ray's useful contribution to the analysis of finite beams on elastic foundation. The tabulated values of influence lines will be of considerable assistance to designers.

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Hetényi's[4] classical solution of the problem of a finite beam on an elastic foundation is well known and it is surprising to read a statement of the author that "Hetényi attempted to solve it analytically." Again, Hetényi has covered a wide range of loading and boundary conditions and Mr. Ray's comment that "His solution is for a concentrated load which is seldom encountered in practice on a long beam" appears misleading. The analysis for influence lines considers a moving concentrated load and this in fact limits Mr. Ray's analytical procedure to the consideration of a concentrated load only.

Mr. Ray's statement that the determination of influence lines for moments, shears, and deflections at 20 sections of the beam requires the expressions to be calculated $3 \times 20 \times 20 = 1200$ times disregards Betti-Maxwell reciprocal relationships! Consider a beam loaded as shown in Fig. B. From reciprocity

$$\Delta_{21} = \Delta_{12} \quad \text{and} \quad \Delta_{31} = \Delta_{13} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Suppose it is required to draw the influence line for deflection at Point 1. It is only necessary to place a unit load at Point 1 and determine the flexure line. This elastic line is the actual influence line for deflection at Point 1. It would therefore appear that only 3×20 calculations are necessary. In the case of a beam of uniform section the advantage of symmetry reduces the number of required computations to 3×10.

From the simple reasoning above it will be obvious that Hetényi's equations for the elastic lines offer an easy means of determining influence lines. In a discussion of the equations expressing deflection, slope, moment, and shear for beams of infinite length Hetényi has illustrated the elegance of the mathematical solution with the statement: "We encounter here the most general applicability of the reciprocal theorem, since it holds in this case not only for displacements, but also for angular deflections, bending moments, and

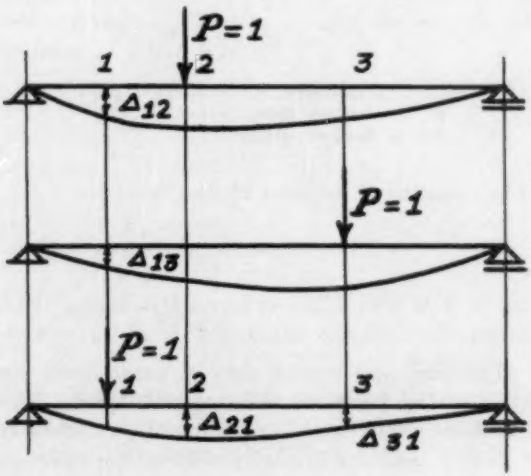

Fig. B—Influence lines for deflection

shearing forces." Since tables are the end products of analytical procedures the designer is not expected to work out expressions containing hyperbolic functions. The labor involved in computing the tables from Hetényi's solutions will compare favorably with that in a finite-difference analysis. Further, by using infinitesimals instead of finite differences a greater degree of accuracy is obtained. Beams with variable cross sections and different end conditions can be analyzed.

The application of numerical procedure to the solution of beams on elastic foundation was first described by Levinton.[5] In a discussion of Levinton's paper Gold[6] illustrated the use of the finite difference method. Wright[7] has used relaxation methods of solution. Several numerical examples using finite differences can be found in a paper by Malter.[8] However, all these numerical methods yield particular solutions which necessitate a repetition of calculations for each new problem.

The use of Fourier series in physical problems, which is usually referred to as harmonic analysis, considerably simplifies methods of solution. Hetényi[4] has applied trigonmetric series to the problem of a beam on an elastic foundation. However the range of applicability of Fourier series is limited by the different end conditions to be considered. Inglis[9] has replaced the sine and cosine functions of harmonic analysis by functions of a more complex nature and developed an analogous method known as the "basic function" method which easily takes into account different boundary conditions. Jaeger[10] has applied the method to the solution of interconnected beam systems. Hendry[11,12] has developed a simple procedure for the application of basic function theory to the analysis of beams on an elastic foundation. The method has been described by the writer.[13]

The differential equation of flexure for a beam on an elastic foundation is

$$EI\frac{d^4w}{dx^4} = q(x)b - kbw \qquad (6)$$

where w = deflection; x = distance along the beam; EI = flexural rigidity; $q(x)$ = intensity of loading; b = breadth of the beam; k = subgrade modulus

The equation is satisfied by the function

$$w(x) = A(\cosh \Theta x + B\cos \Theta x) + C(\sinh \Theta x + D\sin \Theta x) \qquad (7)$$

where $\Theta = \sqrt[4]{kb/4EI}$ —a factor influencing the shape of the elastic line and therefore called the characteristic of the system. A,B,C,D = constants.

The first and second sets of parantheses contain symmetrical and anti-symmetrical functions about the midspan. These functions are termed basic functions. The symmetrical functions will be hereafter referred to as $F_1(x)$, $F_3(x)$, . . . and the anti-symmetrical functions as $F_2(x)$, $F_4(x)$, . . .

TABLE A

	x/l						$\int_{-l/2}^{l/2}[F_n(x)]^2dx$	p_n
	0	0.1	0.2	0.3	0.4	0.5		
$F_1(x)$	−6.52	− 5.58	−2.91	+1.055	+5.77	+10.74	28.1 l	1.506
$F_2(x)$	−0	−24.50	−33.60	−20.15	−11.50	+50.80	698 l	2.50
$F_3(x)$	+174	+80.30	−98.40	−157.60	−12.80	+244.9	15,700 l	3.50

Taking the origin at the center of a beam of length l and applying the boundary conditions at $x = \pm l/2$, $(EI)\ (d^2w/dx^2) = 0$, and $(EI)\ (d^3w/dx^3) = 0$ yields a series of values for the characteristic Θ. The equation has an infinite number of solutions. Putting $\Theta_n = (p_n \pi)/l$, the first two values of symmetrical functions are $\Theta = 1.506\ \pi/l$ and $\Theta_3 = 3.5\ \pi/l$ and the first value of the antisymmetrical functions is $\Theta_2 = 2.5\ \pi/l$.

The applied load $q(x)$ can be expressed as

$$q(x) = \sum q_n F_n(x) \qquad (8)$$

Multiplying both sides by $F_n(x)$ and integrating from $x = -l/2$ to $x = +l/2$ gives

$$q_n = \frac{\int_{-l/2}^{l/2} q(x)\, F_n(x)\, dx}{\int_{-l/2}^{l/2} [F_n(x)]^2\, dx} \qquad (9)$$

The values of $F_n(x)$ and $\int_{-l/2}^{l/2}[F_n(x)]^2\, dx$ have been plotted by Hendry[12] for the first three basic functions. These curves have been reproduced by the writer in Reference 13 and will not be shown here. For the convenience of readers the values are tabulated in Table A.

Noting that the deflection and load distributions are similar the deflection is represented by

$$w(x) = \sum A_n F_n(x) \qquad (10)$$

Substituting Eq. (10) and (8) in Eq. (6) gives

$$A_n = \frac{q_n}{\left(\frac{p_n \pi}{l}\right)^4 EI + K} \qquad (11)$$

The values of A_n thus obtained determine the deflected profile. The moment expressions are obtained by differentiating the deflection expressions twice. The "shapes" of the second differentials of $F_1(x)$, $F_2(x)$, and $F_3(x)$

TABLE B—PRESSURE COMPUTATION

$\frac{x}{l}$	−0.500	−0.475	−0.375	−0.275	−0.175	−0.125	−0.025	0 (origin)
$F_1(x)$	+10.74	+9.5	+4.5	0	−3.7	−5.0	−6.5	−6.52
$F_3(x)$	+244.7	+180	−60	−165	−60	+40	+172	+174
$A_1F_1(x)$	+0.0273	+0.0241	+0.0114	0	−0.0094	−0.0127	−0.0165	−0.0165
$A_3F_3(x)$	+0.0012	+0.0009	−0.0003	−0.0008	−0.0003	+0.0002	+0.0008	+0.0008
$A_1F_1(x)$	+0.0285	+0.0250	+0.0111	−0.0008	−0.0097	−0.0125	−0.0157	−0.0157
Rigid beam deflection, in.	+0.1245	+0.1245	+0.1245	+0.1245	+0.1245	+0.1245	+0.1245	+0.1245
Total deflection, in.	+0.1530	+0.1495	+0.1356	+0.1237	+0.1148	+0.1120	+0.1088	+0.1088
Pressure, ton per sq ft	0.9840	0.9610	0.8720	0.7950	0.7380	0.7200	0.7000	0.7000

have also been plotted by Hendry.[11] Superposition of the deflection and moment values on the corresponding quantities for an infinitely rigid beam gives the final values.

A concentrated load can be analyzed into the series

$$P\sum \frac{F_n(a)\,F_n(x)}{\int_{-l/2}^{l/2}[F_n(x)]^2\,dx} \qquad (12)$$

where a specifies the load position.

The basic function method will be illustrated by application to the author's example on p. 732. The loading $q(x)$ and the deflection $w(x)$ are resolved into the following basic series

$$q(x) = q_1F_1(x) + q_3F_3(x) \qquad (13)$$

$$w(x) = A_1F_1(x) + A_3F_3(x) \qquad (14)$$

The second basic is omitted because it is antisymmetric.

From Eq. (12)

$$q_1 = 0.053 \text{ ton per ft}, \qquad q_3 = 0.00076 \text{ ton per ft}$$

From Eq. (11)

$$A_1 = 0.000211 \text{ ft} \qquad A_3 = 0.000000399 \text{ ft}$$

The computations are tabulated in Table B. Note that the value of 1 ton was taken equal to 2240 lb in converting subgrade modulus and Young's modulus into tons per sq ft, but this does not affect the final values of pressure.

The pressure distribution thus obtained is compared with the influence line solution in Fig. C. The author's solution in Fig. 4 on p. 738 does not appear to be correct. The mean pressure under the beam is equal to $120/(60 \times 2.5) = 0.8$ ton per sq ft. From an inspection of Fig. 4 in the author's paper it is obvious that the area above the mean is much greater than the

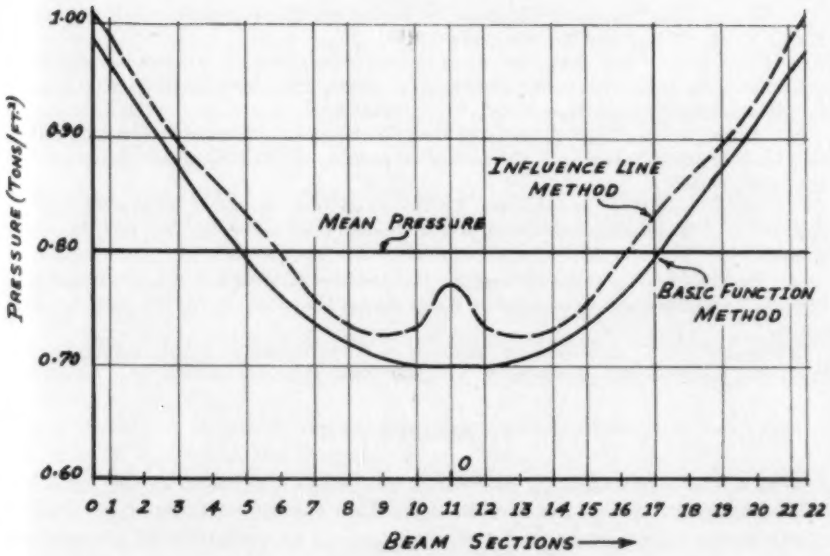

Fig. C—Pressure distribution along the beam

area below it. In fact, an approximate numerical integration of the author's pressure diagram by the writer gave a total upward force of 134 tons whereas the total load = 120 tons. It was therefore necessary to recompute the influence line solution.

The solutions obtained by Hendry using the basic function method retain the parametric form of formal mathematical solutions. This helps the designer in proportioning section suitable. For instance in Eq. (11) the juxtaposition of the subgrade modulus (K) and the flexural rigidity (EI) of the beam will assist in evaluating the relative flexibility of the beam and its foundation. The method is applied easily to beams with variable rigidity.[12] It is sometimes argued that the assumption of a profile will lead to inaccuracies. The degree of exactness can be refined to any extent desired by determining the coefficients multiplying the successive functions of the series using energy concepts and the Rayleigh-Ritz procedure.

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By ANDREW RETI*

This is the third paper to appear in the last few months on the subject of finite beams on elastic foundations.[8,14] This can be considered as evidence of the recent great interest in this question. The author is to be congratulated for attempting to aid the designer by presenting extensive tables, rather than presenting a new mathematical solution to this problem. These tables reduce considerably the bulk of the work involved in computing the pressures, bending moments, and shear forces under concentrated loads acting at the twentieth points of a beam on elastic foundations.

The first attempt to solve the problem of the finite beam with the aid of influence lines was made by Fritz[15] in 1930. Fritz's influence lines give values for only nine positions of the load. Thus, Mr. Ray's tables with 22 positions of the load are an important improvement. It is unfortunate however, that in spite of the title no influence lines are included in the paper. In this way the designer who wishes to use this paper would have to resort to time consuming interpolations if his loading pattern did not fall upon the twentieth points of the beam. Thus, when such interpolations between two or four values became necessary the time saving purpose of the paper would be destroyed.

The parameter m can be expressed in terms of the well known parameter λ used in the articles previously published on this subject. Since

$$m = \frac{Kb}{EI}\left(\frac{L}{20}\right)^4$$

we can transform it into

$$n = \frac{L}{\lambda} = 20\sqrt[4]{m} \qquad \text{with} \qquad \lambda = \sqrt[4]{\frac{4EI}{Kb}}$$

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TABLE C

m	0.0001	0.001	0.005	0.01	0.05	0.1	0.2	0.3	0.4	0.5
$n = \frac{L}{\lambda}$	2	3.55	5.32	6.32	9.45	11.65	13.3	14.8	15.9	16.8
$\frac{n\lambda - n\lambda_{-1}}{n\lambda_{-1}}$, percent	77.5	49.5	18.8	50	24	14	11	7	5	

The value λ is called "characteristic length" of the beam by Hetényi and others. The writer prefers the term "elastic length" or better yet "the unit of elastic length" of the beam. This parameter has a geometric significance which can be easily visualized, while the parameter m has no such significance attached to it. The λ is the length of a freely supported beam (of the same cross section) which under a concentrated load applied at midspan would be stressed by the same bending moment as the infinite beam on an elastic foundation.

From the above it follows that the values of the parameter m can be substituted by $n\ L/\lambda$ as shown in Table C.

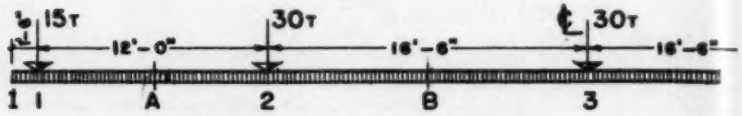
The last line of this table shows the growth of the parameter n. It points out that although the author's tables include a wide range of stiffnesses, the most useful range ($2 < n < 8$) is only covered by four columns. It is highly recommended that this part of the table be extended by including three or four intermediate columns.

Mr. Ray's statement that a beam with an m value of 0.0001 can be considered infinitely stiff confirms this same fact established by Kogler[16] in 1937. For short beams whose length is smaller than two "elastic lengths," $L \leqq 2\ \lambda$, the influence of the boundary conditions (free ends) is predominant. For medium size beams $3 < n < 5$ the influence of the end conditions is appreciable only in the end thirds of the beam. For long beams, $n > 6$ the center part of the beam can be considered as an infinite beam. The value of λ is essential in the design of economical foundation beams and its value should be related to the average distance between the external loads. Such relationships would be difficult to establish using an abstract parameter such as m.

The numerical example presented in the paper illustrates the use of the tables very well. The picture would change radically if the conditions in a problem did not correspond to those used in the tables. The risk of errors due to the cumbersome interpolations required would discourage many people from using these tables. It is inevitable that sooner or later a series of complete influence lines will be published to fill the gap.

The general solution of the problem of a finite foundation beam was given by Bleich[17] in 1938. A modified semigraphical variant of Bleich's method was published by the writer.[18,19] A short description of this method was presented in the writer's discussion of ASCE *Proceedings* Paper No. 1722.[20]

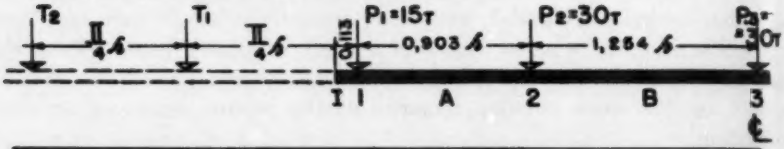
The writer's method gives results with the accuracy of three decimal places. The most complicated and irregular cases are quickly solved utilizing only three influence line diagrams.



$I = 7{,}82$ FT $E = 3{,}10^6$ PSI $K = 100$ LB/IN^3 $B = 2{,}5$ FT

$M = 0{,}01$ $\delta = 158{,}5^2 = 13{,}29'$ $N_{1,2} = \frac{12}{13{,}29} = 0{,}903$ $N_{2,3} = \frac{16{,}5}{13{,}29} = 1{,}254$

THE ELASTIC SCHEME OF THE BEAM



	PRESSURE (1000 AN)						B MOMENT (1000 CN)						PT
	I	I	A	2	B	3	I	I	A	2	B	3	
T2	+220	+178	+45	-20	-45	-31	-208	-205	-165	-109	-40	-3	+17,0
T1	+652	+585	+320	+140	-2	-43	—	-55	-195	-200	-128	-53	+51,4
1	+980	+1000	+845	+560	+235	+36	+788	+1000	+290	-110	-207	-160	+15,0
2	+520	+590	+845	+1000	+750	+368	-110	-60	+290	+1000	+115	-180	+30,0
3	+20	+48	+160	+360	+750	+1000	-148	-162	-205	-180	+115	+1000	+30,0
4	-39	-41	-41	-18	+100	+368	-18	-24	-60	-114	-192	-180	+30,0
5	-16	-18	-30	-41	-32	+36	+8	+7	-2	-23	-80	-160	+15,0
T3	—	—	-9	-18	-35	-43	—	—	+9	+7	-9	-53	+51,4
T4	—	—	—	—	-16	-31	—	—	—	—	+9	-3	+17,0
Σ	+667	+658	+578	+543	+482	+479	0	+1,51	-7,13	+7,11	-10,7	+8,9	Σ
P	1,0	0,99	0,87	0,82	0,72	0,718	0	+5,02	-23,7	+23,6	-35,7	+29,6	M

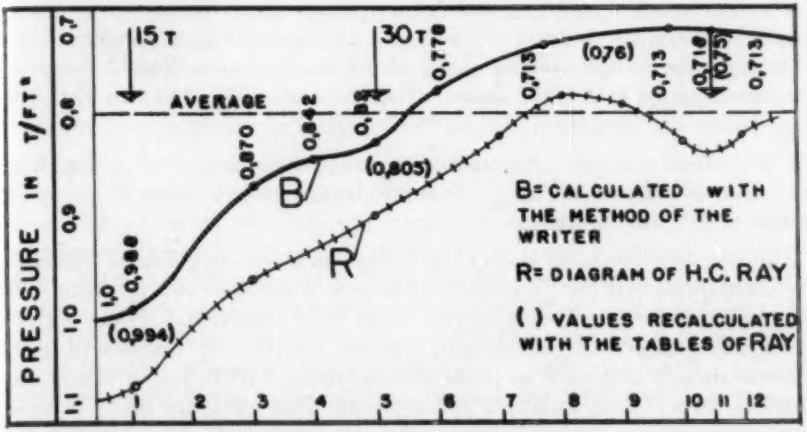
	SHEAR (1000 DN)						PT
	I	I	A	2	B	3	
T2	—	+18	+63	+65	+41	+17	+17,0
T1	-322	-260	-60	+35	+66	+48	+51,4
1	+890	+1000	-570	-250	-15	+63	15,0
2	+200	+250	+570	+1000	-430	-90	30,0
3	-67	-63	-20	+90	+430	+1000	30,0
4	-28	-32	-52	-65	-47	+90	30,0
5	-4	-5	-16	-32	-63	-63	15,0
T3	—	—	—	-6	-22	-48	+51,4
T4	—	—	—	—	-3	-17	+17,0
Σ	0	+6,54	+5,14	+24,0	+1,56	+30,0	
Q	0	+3,27	+2,57	+12,0	+0,78	+15,0	

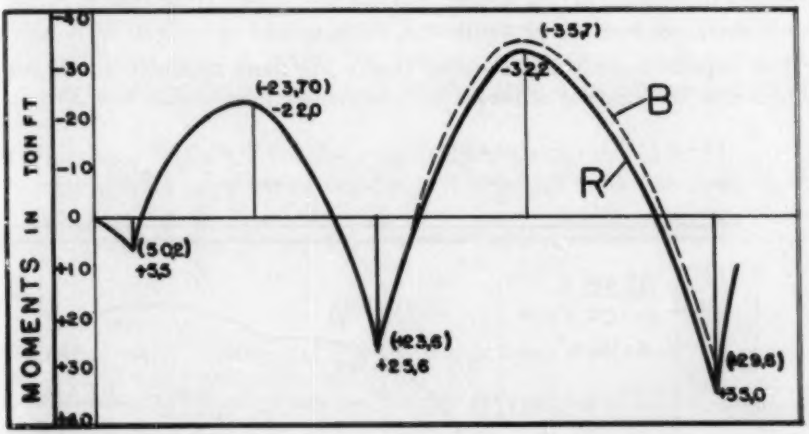
$$P = \frac{1}{2\delta B} \Sigma\, P_i A_N \left(T/FT^3\right)$$

$$M = \frac{\delta}{4} \Sigma\, P_i C_N \left(T\,FT\right)$$

$$Q = \frac{1}{2} \Sigma\, P_i D_N \left(T\right)$$

Fig. D—Numerical example of K. C. Ray solved by writer's method





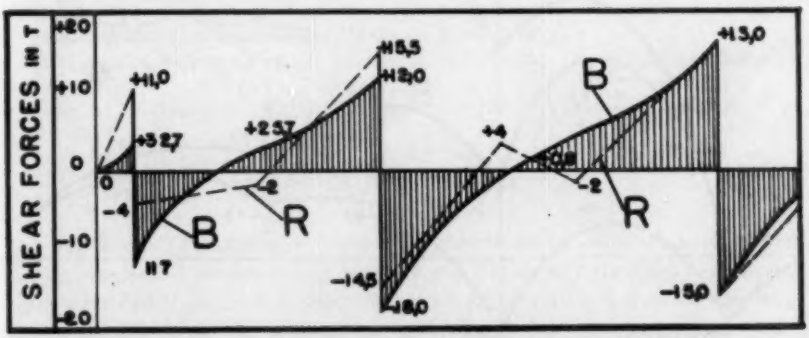


Fig. E—"Corrected" diagrams for numerical example

The numerical example given in Mr. Ray's paper shows very well the danger of using his tables. The pressure diagram in Fig. 4 appears to be in error since the average pressure line of 0.8 ton per sq ft should divide the pressure diagram into equal areas. The shear diagram shown in Fig. 6 also shows some inadequacies.

The writer has applied his method to the problem as shown in Fig. D and the results are shown in Fig. E. It would be advisable to recheck some of the figures of Mr. Ray's tables.

The so-called "classical theory" of beams on elastic foundation is based on the assumption that the "coefficient of subgrade reaction" is a constant. This assumption is far from being correct under field conditions. Some engineers modify this theory by taking into account the various theories of subsoil pressure distribution such as those of Boussinesq, Fröhlich, and others. The classical theory thus modified yields results that compare more favorably with conditions encountered in practice. The computations required to solve a practical problem by the second method are so extensive that in practice many designers hesitate to use it as a working tool.

The imperfections of the classical theory are most apparent in the result obtained in the analysis of beams with extremely unfavorable boundary con-

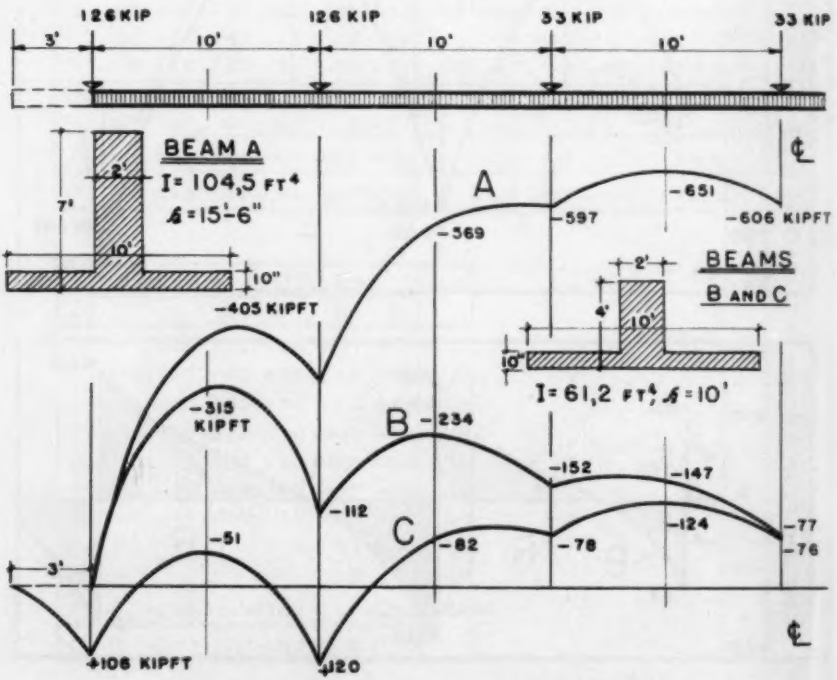


Fig. F—Influence of beam rigidity and projection

ditions and supporting a uniformly distributed external load. These calculated results are in striking contradiction with the results of field observations, measurements made on completed foundation beams, and of experimental investigations. Thus the unlimited application of the classical method without engineering judgement can lead to serious mistakes.

An example of an inconsistent design resulting from excellent mathematical analysis untempered by engineering judgement is shown in the bending moment diagram of Fig. F. The "free end" condition and the large concentrated load applied near the end of the very stiff beam produced a moment diagram that forced the designer to reinforce this beam heavily all along the upper edge without having to reinforce it at the bottom. Yet reducing the depth of this beam from 7 ft to 4 ft (Beam B) and thus making it more flexible produced bending moments whose magnitude is only one-fourth of those found in the stiffer beam. If in addition we added a small extension at each end of the beam the picture would change completely (Beam C).

This example is sufficient to show that new mathematical tools for the analysis of foundations are not enough in themselves. It is also necessary to supply additional rules of thumb for the general layout of the foundations and guides for the proportioning of such features as ribs, projections, pedestals, and haunches.

The influence tables presented by the author are helpful in reducing the mathematical work involved in the design. This will allow the designer to direct his attention to the over-all problem of the foundation design which is more important than the mechanics of the calculation.

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By **DONALD A. SAWYER***

This paper was of particular interest to this reviewer because he had been developing a Bell-2 program for use on the IBM 650 electronic computer which is intended to grind out the "long expression with hyperbolic functions" presented by Hetényi. This program has been put to use several times and

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TABLE D—COMPARISON OF VALUES

m	Area [Eq. (19)]	Author's average area	Average percent error
0.5000	0.1000	0.0581	41.9
0.4000	0.1250	0.1249	0.1
0.3000	0.1667	0.1884	13.0
0.2000	0.2500	0.2501	0.0+
0.1000	0.5000	0.4990	0.0+
0.0500	1.0000	2.7586	175.9
0.0100	5.0000	4.9819	0.2
0.0050	10.0000	27.3768	173.8
0.0010	50.0000	49.8624	0.3
0.0001	500.0000	55.6637	88.6

has afforded an excellent opportunity to compare the results of the two methods. Unfortunately, the results of the finite difference method could not be verified.

Investigation shows that the tables in Mr. Ray's paper contain many discrepancies and inconsistencies which must be corrected or explained before they can have much value. Some of these errors may have resulted from unfortunate typographical mistakes, but others appear to be more basic. Briefly, the following points need further explanation:

1. Considering the approximate method used to obtain the values in the tables, it appears that more significant figures than are justified have been used. To illustrate this, one might browse through the tables using Maxwell's law of reciprocal deflections. Some values check exactly, whereas others fail to agree in the second significant figure.

2. There appears to be a decimal error for the $m = 0.0001$ columns. This would indicate that the "area" figure given at the end of each of these columns should be closer to 555, or thereabouts, rather than the 55.5- - given. It will be shown in the next item of discussion that even this will not fully correct the error.

3. If the numbers given are correct, one could integrate the pressure over the entire area of the base and obtain the total load P which the beam supports. Hence, one can write,

$$P = \int_0^L (pb)\, dx \qquad (15)$$

where: p = unit pressure on base, psi
b = width of beam in in.
L = length of beam in in.

From the paper

$$p = (\text{I.O.}) \times K \times P \times \left(\frac{L}{20}\right) 3 \times \frac{1}{EI}$$

where: I.O. = influence ordinate
K = subgrade modulus, lb per cu in.
EI = beam flexural rigidity, lb-in.^{2}

Thus

$$P = P \int_0^L \frac{Kb}{EI} \left(\frac{L}{20}\right)^3 (\text{I.O.})\, dx \qquad (16)$$

But since

$$m = \frac{Kb}{EI}\left(\frac{L}{20}\right)^4$$

one can write

$$P = mP\left(\frac{20}{L}\right)\int_0^L (\text{I.O.})\,dx \quad \ldots\ldots (17)$$

The "area" which the author gives at the end of each column of figures could only mean

$$A = \frac{1}{L}\int_0^L (\text{I.O.})\,dx \quad \ldots\ldots (18)$$

since the tables could be thought of as being for a beam of unit length. Hence, in Eq. 17

$$\int_0^L (\text{I.O.})\,dx = L\,A \quad \ldots\ldots (18a)$$

Finally, then, one obtains

$$\text{Area} = \frac{1}{20\,m} \quad \ldots\ldots (19)$$

Eq. (19) relates the area of the influence line to the value of the parameter m. Table D lists the theoretical values [as determined from Eq. (19)] of the area for various values of m as compared to the author's actual values of the area. Note that the magnitude of the average error varies widely. Some of the values are too extreme to be of any use. The possibility of a decimal error for the $m = 0.0001$ case is clearly indicated.

4. The author states that a beam with an m value of 0.001 may be considered as an infinitely stiff beam. The writer agrees with this. However, the tables of values for the deflections of such a beam, as given by the author, have disturbing implications. Refer to Table 11, for example. This indicates that an infinitely stiff beam loaded by a single concentrated load in the center will experience a deflection such that the center deflects about three times as much as the ends. This would indicate that such a beam has considerable flexibility. Thus, the variation of deflection along the length of the beam is questionable. It is of interest that the solution given by Hetényi indicates that a beam of comparable stiffness, i.e., $m = 0.0001$, would deflect about 1.12 times as much in the center as at the ends.

5. The curves presented in the example problem appear to be in considerable error. A shear diagram such as shown in Fig. 6 could not possibly result from the load distribution shown in Fig. 4.

6. Question is raised of the use of photoelasticity as an experimental check. Basically, the photoelastic method would yield a somewhat crude stress distribution. From this, one might obtain either deflections or load distribution by numerical integration or numerical differentiation, respectively. Either procedure would seem too crude to provide the excellent check reported. Would the author care to elaborate on his experimental technique?

7. Though this discussion may seem severe, many of the comments regarding high accuracy are largely academic since the true value of the subgrade modulus, K, may not be known within 50 percent for practical cases of application.

The author and his associates have undoubtedly expended much effort in the computations leading up to the tables presented. Obviously, several 20 x 20 matrices had to be inverted. It may be that the author is not as fortunate as the writer in having an electronic computer at his disposal. A program designed to solve the equations presented by Hetényi is fairly simple to write and the solution on the machine is reasonable fast. For example, the machine will compute the deflection and bending moment at the tenth points (including the two end points) for a single concentrated unit load placed successively at each of the tenth points. In addition, it will compute the influence line for the end reaction of a finite beam on an elastic foundation with one end on a nonyielding pin support. All of this takes about 2 min of machine time per beam length. Beam lengths are entered in terms of the dimensionless parameter λL, which corresponds to the m given by the author. Since the Bell-2 coding method operates in "floating point" arithmetic (i.e., scientific notation), the program will convert the results of the main computation into standard fixed decimal numbers at an expense of about 2 extra min of machine time per beam length. The writer will be happy to provide this program to anyone desiring it at the cost of reproduction and postage.

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AUTHOR'S CLOSURE

The author thanks those who have so ably discussed the paper, and offers below clarification of some of the points raised by the discussers.

Prof. Pickett's suggestion to use modulus values for interior loading is welcome.

The statement by Mr. Gedizli that the fundamental equation holds true only in the case of an infinitely long beam appears to be not in order. The equation should hold good in all cases irrespective of boundary conditions. The method suggested by him is, however, considered useful for applying to

TABLE E

Sections	1	2	3	4	5	6	7	8	9	10	11
Pressure, tons per sq ft	0.9950	0.9400	0.8950	0.8610	0.8320	0.7940	0.7590	0.7380	0.7320	0.7360	0.7740
Bending moment, ft-tons	−5.4450	23.3100	25.0560	21.9870	−25.9380	11.1240	30.4020	33.3630	22.4370	−2.7450	−35.1000
Shear, tons	11.3850 −3.6150	4.3500	−2.2770	−9.2460	15.2310 −14.7690	9.4650	3.9120	−1.6290	−6.7980	−12.2970	15.0000 −15.0000

a statical load system, but for a beam subjected to rolling loads the answer is difficult. Regarding computations, it is regretted that some arithmetical errors crept into the values of pressure, moments, and shears in the example. These should now check with Mr. Gedizli's values. The corrected table is given as Table E.

The tables may be used for all cases. When there is any discontinuity in sign of subgrade reactions, the length of the beam is to be adjusted accordingly. This is analogous to continuous beam problems.

Dr. Reti's observations with respect to drawing influence lines is aptly realized by the author. In this connection he is of the opinion that provision of ordinates for curves lends a better and more correct picture. The author had in mind, while preparing this paper, the useful tables for continuous beams by Griot. As suggested, the values of pressures, bending moments, and shears have been rechecked and these are now in order and satisfy the observations of Dr. Reti. It is mentioned here that the problem was inserted to illustrate the use of the tables only and the results were obtained from a designer's point of view. It is agreed that subgrade modulus K considered as a constant may not give a correct picture of foundation behavior, but as a ready tool for mathematical computations, the tables should prove extremely useful.

Mr. Reddy is too critical in his comments. Hetényi definitely has solved the problem, but his solutions are too difficult to apply in the case of a moving load. The problem originated in designing a raft foundation carrying an underground railway and it was impossible to find positions for the worst loading for design. Hence these influence lines were worked out. Reduction of the number of equations to only 3×10 is not correct. For a finite beam, half of the beam, i.e., 10 points, has to be loaded to get the true picture, or with Maxwell's reciprocal theorem applied each time the equation would reduce by 1 until finally the number vanishes. This would give an A.P. starting from 20 reducing to 0. With 10 different values of m the total number of equations would be 5760. Whatever may be the method, the influence lines are there and let them stay. The corrected values of pressure, moment, and shear distribution would dispel other doubts of Mr. Reddy. The new value of upward pressure works out to 124 tons instead of 134 tons, showing therein that the deviation is well within 4 percent.

Mr. Sawyer must have applied the check both to interpolated and actually worked out figures. This will account for the discrepancy while applying Maxwell's reciprocal law. Regarding the shifting of the decimal point as suggested for area, values for ordinates for $m = 0.001$ may be compared with those for $m = 0.0001$. This will at once suggest that the area appearing in the column is in order. Thanks are due to Mr. Sawyer for checking the areas, where cases for m having values of 0.001, 0.01, 0.1, 0.2, and 0.4 are in complete accord with calculated values. Interpolated values for m have shown variations in areas. Interpolations have been done on a linear basis as mentioned in the body of the text; the errors are definitely due to this. The above statement will satisfy Mr. Sawyer's other points. Regarding the experiment, this was part of finding out stresses in a spread foundation, only the deflection value had been checked. There seems nothing to elaborate. It is regretted that the shape of the shear diagram as plotted was not in order although values were correct. Finally, as observed by Mr. Sawyer, with all the uncertainties of actual conditions, the tables presented will provide for at least the nature of the flexural profile and near possible design values for the designer, particularly when confronted with the cases of rolling loads.

ACI News Letter

30, No. 12 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1959

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- **Concrete Pavements and Concrete Bases** (ACI 617-58)
- **Precast Concrete Floor and Roof Units** (ACI 711-58)
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ACI

News Letter Contents

June 1959

On the Cover—Aerial view of the Abidjan Bridge, on the Ivory Coast of French West Africa. Main span is of precast, prestressed concrete. Access ramp of Petite Bassam Island is in foreground. See story on p. 5.

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Hot Weather Concreting . . .

Lightweight Concrete Proportions . . .

Measuring, Mixing, and Placing Concrete . . .

New Standards

Three standards adopted by the American Concrete Institute at its 55th annual convention in Los Angeles have been ratified by letter ballot of the Institute membership; two are new and one is a revision of an existing standard. This increases the current list of ACI standards to 15.

"Recommended Practice for Hot Weather Concreting (ACI 605-59)" was prepared by ACI Committee 605 under the chairmanship of Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.

"Recommended Practice for Proportioning Lightweight Aggregate Structural Concrete (ACI 613A-59)" was a part of the work of ACI Committee 613 and prepared by the subcommittee on proportioning lightweight aggregate concrete, with J. J. Shideler, manager, Products and Applications Section, Portland Cement Association Research and Development Laboratories., Skokie, Ill., serving as committee chairman.

The revised standard "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)" supersedes the 1942 standard. It was prepared by ACI Committee 614 under the chairmanship of Lewis H. Tuthill, concrete engineer California State Department of Water Resources, Sacramento.

Hot weather concreting

"Recommended Practice for Hot Weather Concreting (ACI 605-59)" provides data useful in minimizing detrimental effects of hot weather on concrete. Means are described for reducing

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ACI Publications Department

concrete temperature by proper attention to ingredients; methods of production and delivery; and care in placement, protection, and curing. Information is given on the use of admixtures to reduce mixing water requirements and to retard setting. Emphasis is given to the importance of meticulous attention to the use of standard procedures in testing concrete made in hot weather.

The problems of concreting in hot weather have increased in recent years due to a number of factors including more finely ground and more rapidly hardening cements, handling of larger batches of concrete, use of thinner sections, and increased speeds in all construction operations. This standard suggests procedures for reducing the hazards of hot weather in the more usual classes of construction.

Lightweight concrete proportioning

"Recommended Practice for Proportioning of Lightweight Aggregate Structural Concrete (ACI 613A-59)" is intended as a supplement to "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" and describes a procedure for proportioning structural grade concrete containing lightweight aggregates. This procedure does not require the use of values for specific gravity or absorption of the aggregates but utilizes a "specific gravity factor." Use of this factor is illustrated and examples are included for proportioning both air-entrained and non-air-entrained mixes.

Measuring, mixing, and placing

The revised standard, "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)" gives special consideration to practices which contribute to maximum uniformity, homogeneity, and quality of concrete in place. To portray more clearly certain of the principles involved in these practices, illustrations of good and bad practice are included as a part of this recommendation.

In making these revisions the committee was guided by the belief that progress in improvement of concrete construction practices will be better served by endorsing a comparatively high standard of practice rather than "common practices." Recommendations are therefore made with the thought of permitting the user flexibility in his specifications to the extent he considers worthwhile for his particular project. Many of the practices recommended are included primarily to improve concrete uniformity. Where the methods described have been carried out properly, the effort and investment have usually been rewarded by concrete of higher quality, a smoother operation, and higher production rates, all factors which tend to offset any additional cost.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

John L. Rohwedder
Corps of Engineers
U. S. Army
Rock Island, Ill.

Committee 212, Admixtures

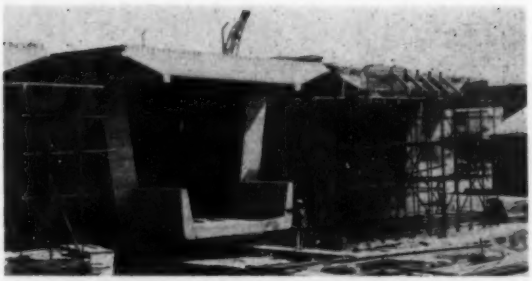
Robert F. Adams
California Department of Water Resources
Sacramento, Calif.

Committee 337, Strength Evaluation of Existing Structures

A. S. Neiman
Headquarters, U. S. Air Force
Washington 25, D. C.

Committee 340, Ultimate Strength Design Handbook

J. J. O'Sullivan
The Rand Corporation
Santa Monica, Calif.



On-shore casting yard. Completed girder section on left about to be slid onto barges for placing in the structure

Bridge at Abidjan

Until 1957 the town of Abidjan, capitol of French West Africa, was connected to its deep water harbor on Petite Bassam Island by a floating bridge. Growth in that area had made the structure inadequate. In May, 1957, a modern structure was opened that relieved the traffic problems and overcame the foundation difficulties of that Ivory Coast area. The new bridge currently handles 25,000 vehicles per day, 2600 per hr during peak periods. An aerial view of the bridge and surrounding area appears on the News Letter cover.

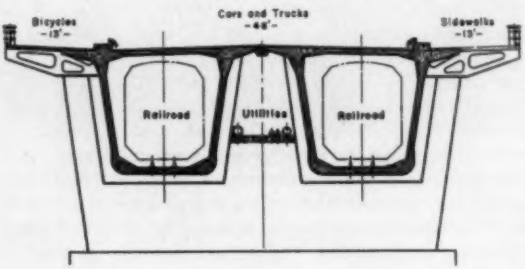
Precast girders

The new precast, prestressed bridge spans 1220 ft over water with two 292-ft approaches. The main span is built of eight spans of twin trapezoidal girders. Traffic loops, designed to eliminate traffic crossing, give the roadway a total length of 3940 ft. Vehicular, bicycle, and foot traffic are carried across the top of the hollow girders on separate roadways. Railway traffic flows through the girders and utilities are carried through a duct formed at the junction of the two girders by a precast slab.

Each of the 16 girder sections of the main bridge was cast on shore and floated to its final position, The 800-ton sections were prestressed with 30- and 100-ton Boussiron-BBR prestressing cables. A number of extra ducts were cast into the sections to provide for additional prestressing in the future.

Foundation problems

Soil explorations indicated that at the center of the lagoon to be crossed the water depth was 32-39 ft. The material beneath the water was an unsuitable muck which existed to a depth of 65-78 ft, and compact silt 55-59 ft deep. Beneath the silt were various sands with small gravel interspersed with thin strata of hard clay. Suitable foundation material was found at 164-180 ft below water level. Because of foundation conditions it was decided to support the bridge on caissons. The hollow caissons,



Cross section of Abidjan Bridge. Main portion is built of twin trapezoidal girders precast in sections

Girder section, weighing 800 tons, being floated to bridge site

23 ft wide by 19½ ft high, 152½ ft on centers, are surmounted by piers forming a double support which reduces the span of the bridge girders to 134½ ft.

Footing piles

After several approaches to the foundation problem, it was decided to make the caissons simple load-bearing footings. Each pier has eight caisson piles, 4 ft 9 in. in diameter. Four of the piles are vertical and four are battered 15 percent; all penetrate the sand strata about 50 ft. The piles are precast, prestressed hollow units placed in holes bored into the soil with Béneto drilling equipment. The piles are made of pozzolonic and slag cement concrete to resist the action of the acid in the underlying muck. A bituminous coating was used on them above the sand strata as additional protection.

Additional frictional resistance was provided by plastic sleeves at the bottom of the piles into which grout was pumped forming a "bulb" at the pile end.

The project was designed by Entreprises Boussiron, Paris. Its African subsidiary, Société d'Étude et de Travaux Pour l'Affrique Occidentale, Abidjan, in cooperation with Société Française d'Entreprises de Dragage et de Travaux Publics, Abidjan, were the contractors.

ASTM schedules annual meeting for June 21-26

Among the 36 technical and report sessions scheduled for the ASTM 62nd annual meeting at Chalfonte-Haddon Hall, Atlantic City, N. J., four symposiums will deal exclusively with cement, concrete, and road and paving materials.

ACI member L. J. Minnick, G. and W. H. Corson, Inc., Cheltenham, Pa., will open the session on concrete, June 22, with a discussion of fundamental characteristics of pulverized coal fly ashes.

Other ACI members appearing on the program include Bryant Mather, U. S. Army Engineer Waterways Experiment Station, Jackson, Miss., who will present a joint paper with L. Pepper on the effectiveness of mineral admixtures in preventing excessive expansion of concrete due to alkali-aggregate reaction; E. C. Higginson and G. B. Wallace, U. S. Bureau of Reclamation, Denver, will present a paper on control testing for separation of lightweight material from aggregate.

At the afternoon session on concrete, I. A. Benjamin and G. D. Ratliff, Granco Steel Products, Co., Granite City, Ill., will present a paper on an in-place strength test for low-density concrete. C. E. Kesler, University of Illinois, Urbana, will conclude the session with a discussion based on an investigation of the effect of length to diameter ratio on compressive strength of different concretes.

ACI members speaking at the session on cement, June 25, include: A. D. Conrow, Ash Grove Lime and Portland Cement Co., who will open the session with a paper on an approach to the problem of evaluation of plasticity and workability of pastes and mortars. Bailey Tremper, California Division of Highways, Sacramento, will speak on the control of gypsum in portland cement; and J. R. Dise, National Bureau of Standards will discuss the Cement Reference Laboratory.

Museum in Concrete:

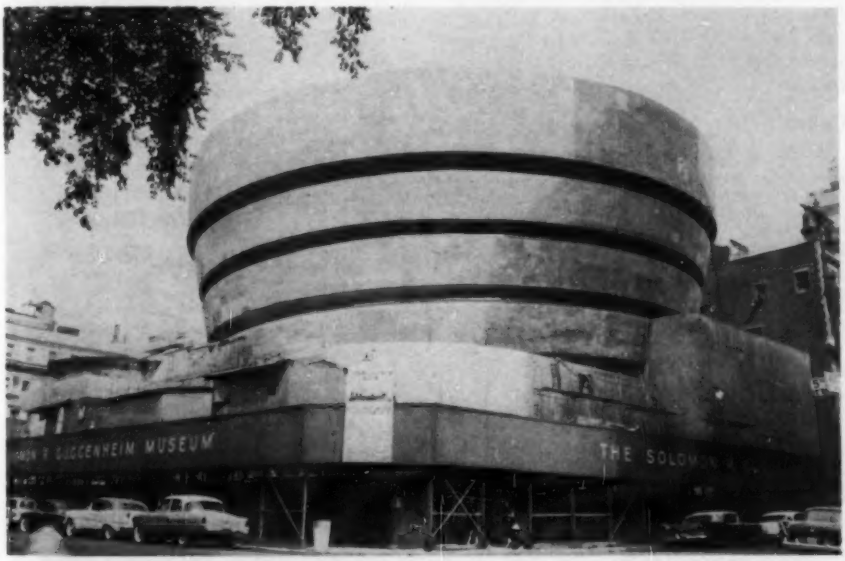
no expansion joints required

SPIRALLING CONCRETE, expanding as it ascends, houses exhibition galleries at the Solomon R. Guggenheim Museum across from Central Park on Fifth Avenue in New York. The $3 million dollar building was designed without expansion joints so that it will expand and contract as a unit.

Two circular towers at the museum's north and south ends rise above a 201 x 128-ft rectangular first floor. A variety of curved concrete surfaces comprise both interior and exterior walls, ramps, slabs, and columns. Six levels of the south exhibit space in the larger tower are connected by a ramp which corkscrews upward toward a glass dome in the roof, lighting the main gallery on the first floor. A circular room is cantilevered out from the ramp of the wing's second level. The ramp, 16 ft wide at the first floor level, determines the exhibition wing's shape as it spirals outward and upward at a 3 percent slope, broadening to 36 ft at the sixth level. Administrative offices are housed in the 54 ft diameter, tri-level north wing.

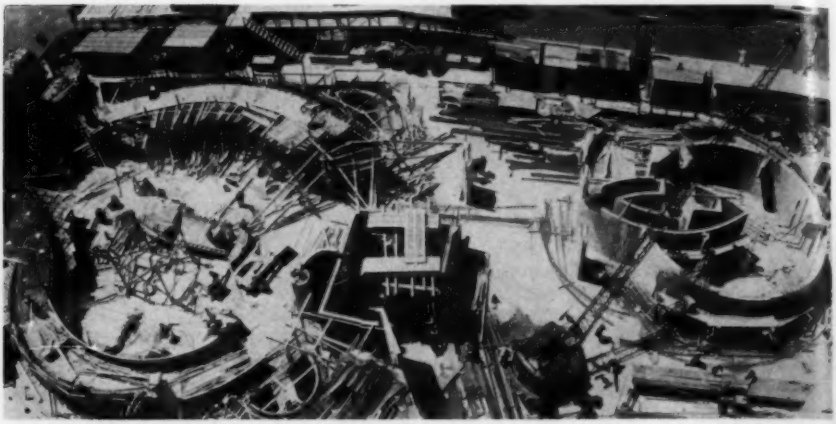
Precision formwork demanded

Careful concrete control and exacting measurements contributed to the building's smooth exterior surface, sharp corners, and precise final measurements. Architectural features such as curved wrought iron doors and windows which fit directly against the concrete frame called for near-perfect ac-



New York Herald Tribune photograph

South wing of the museum as it neared completion. The six levels inside are connected by spiral ramp which rises at a 3 percent incline, increasing from 16 ft wide at the first level to 36 ft at the sixth. Top of circular north wing is barely visible at left



Construction of the north (extreme right) and south circular "wings" of the Solomon R. Guggenheim Museum demanded painstaking accuracy in forming intricate curved surfaces. No expansion joints were included in building design

curacy in laying out the formwork. Engineers worked alongside the carpenters. Only a third of the 4 x 8-ft plywood panels were used full size; the rest had to be cut into smaller pieces. The building has few straight lines, and Masonite as thin as ¼ in. was used for surfaces of small radius; plywood served to form other curved portions of the building.

Concreting in hot weather

Much of the structure above ground level was concreted during the hot summer of 1957. Specifications called for a 3500-psi lightweight concrete with a 3- to 4-in. slump. Fourteen ounces of a densifying retarder was added to each cubic yard of concrete during the hot weather in an attempt to overcome usual warm weather problems of cold joints, loss of slump, and honeycombing. The contractor reported that shrinkage cracks were entirely eliminated.

Ease of placement was especially important in sections such as the cantilevered room on the south wing's second level. The admixture kept the concrete from hardening long enough for it to be vibrated into place.

Each day's concreting varied from 40 to 200 cu yd. The circular wings were concreted one level at a time. For each ramp level an average of three separate concreting operations were required, and ten individual ones for the 11 ft high interior wall. Ramp formwork was supported directly on the level below. The 5-in. exterior wall, which rests on a slab connected to the ramp, was placed pneumatically from the interior against curved plywood forms. About 500 cu yd of concrete were required for each level in the south wing, and 200 cu yd in the north wing.

Architect for the structure was the late Frank Lloyd Wright, and general contractor was Euclid Contracting Corp., New York.

BRI announces election of 1959 officers

Building Research Institute conducted its 8th annual meeting at Pittsburgh on April 7-8 with more than 675 in attendance.

Results of the election by the Board of Governors were announced: Harold L. Humes, Baldwin-Hill Co., New York, will be president; and Robert W. Cutler, Skidmore, Owings and Merrill, New York, vice-president. The newly elected officers will take their chairs effective July 1, with Charles H. Topping, senior architectural and civil consultant, E. I. duPont de Nemours and Co., Inc., retiring at that time after 2 years as president.

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Another notable example of the trend toward reinforced concrete construction is the new "Addition to the Brown Palace Hotel" in Denver, Colorado. Important reasons for utilizing reinforced concrete construction for this handsome new structure were speed of construction and flexibility of design. The inherent fireproof qualities of reinforced concrete make lower insurance rates possible. The ready availability of reinforcing steel permitted the job to get under way at once. Before you build, investigate this economical, flexible, fire-safe, and timesaving medium of construction.

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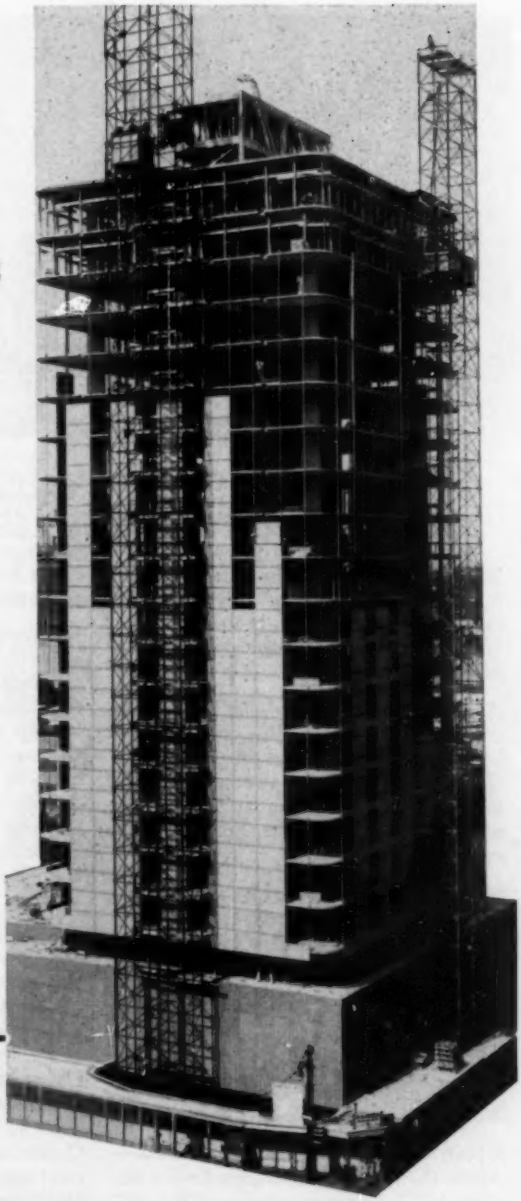
Structural Engineer: Seelye, Stevenson, Value & Knecht, New York City
General Contractor: N. G. Perry Construction Company, Denver, Colorado



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Positions and Projects

Plans for ACI Mexico City regional meeting shape up

With the selection of the local committee, plans are rapidly taking shape for the 12th ACI regional meeting at the Continental Hilton Hotel, Mexico City, November 3-5.

Alberto Dovali Jaime, Universidad Nacional de México, is general chairman of the local planning committee with Adolfo Zeevaert, Torre Latino Americana, serving as secretary, and Antonio Robles, de Cemento Mixcoac, treasurer of the group.

Leonardo Zeevaert, consulting engineer, is in charge of the technical program which will feature concrete problems and construction techniques of prime importance in Mexico and southwestern United States.

Balance of the planning committee is comprised of Enrique Torres de Alba, La Consolidada, S. A., publicity; Alfonso Marín E., civil engineer, membership promotion; Felix Candela, architect, inspection trips; A. Pardo Boland, consulting engineer, exhibits and displays; Luis Flores Arias, civil engineer, finance; and Francisco Santos Oliva, civil engineer, educational contacts.

Rafael Hernandez Hernandez, civil engineer, is in charge of social events and Señora Marcela Castro Villagranada de Hernandez is chairman of the ladies entertainment committee.

California chapters cooperate in concrete conferences

The ACI northern and southern California chapters cooperated with the Institute of Transportation and Traffic Engineering, University of California, and the University Extension in presenting the fourth series of conferences on practical concrete problems. The conferences were conducted in San Diego, April 29 with 125 in attendance, and in Los Angeles, April 30 with 265 attending.

ACI Vice-President Joe W. Kelly, professor of civil engineering, University of California, Berkeley, served as moderator at both conferences.

R. C. Dorland, structural engineer; M. F. Landon, M. H. Golden Construction Co.; P. W. Helsley, Testing Engineers; W. P. Kemper, Jr., Pre-Mixed Concrete Co., all of San Diego; and R. E. Tobin, Portland Cement Association, Los Angeles, made up the panel for the San Diego conference.

Samuel Hobbs, Southern California Chapter, ACI; E. R. Wohl, Emil Wohl Construction Co.; Ernst Maag, State Division of Architecture; R. W. Spencer, Southern California Edison Co.; and R. E. Tobin, all of Los Angeles comprised the panel for the Los Angeles conference.

ACI Oklahoma chapter features slab demonstration

The May 16 meeting of the ACI Oklahoma chapter featured a demonstration on problems of slab placing and finishing. The demonstration was staged at the Livestock Pavilion at the Oklahoma State Fairgrounds, Oklahoma City.

The demonstration covered placing of high- and low-slump concrete, concrete of different qualities, air-entrained and non-air-entrained concrete. Plastic shrinkage was forced on a portion of the job with hot air blowers. Approximately half the slabs were placed 7 days prior to the demonstration to show differences in shrinkage and surface conditions. The remainder of the slabs were placed early on the morning of the demonstration and were ready for finishing during the meeting.

An invitation to attend the demonstration was extended to local finishers, home builders, contractors, architects, engineers, government officials, cement company representatives, and ready-mixed concrete companies.

Institute headquarters attracts visitors

Interest in the new Institute headquarters in Detroit continues to mount. During April over 200 engineers, architects, and students inspected the building.

The largest groups to include the Institute building in their itinerary were 70 German engineers and architects on a country-wide tour; about 50 Michigan State Univer-

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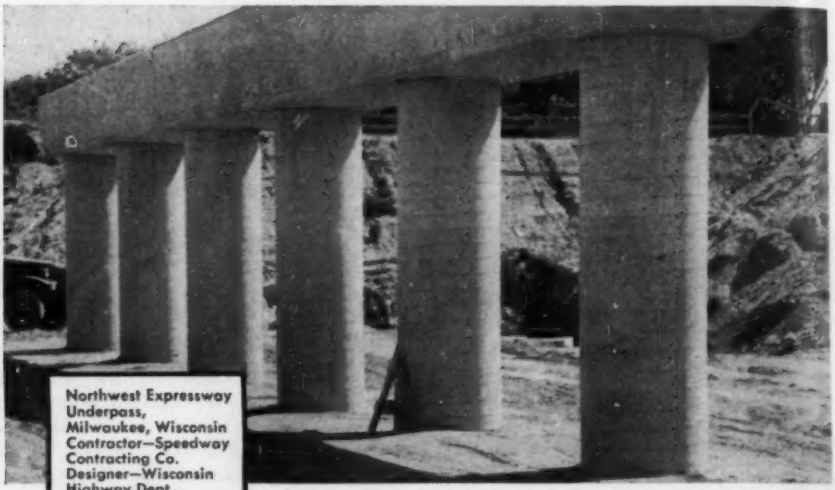

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sity students; and approximately 40 Pennsylvania State University students from the Department of Architecture.

World-wide interest is evident as the ACI visitors' register shows guests from Sweden, England, Poland, Germany, and Canada among the recent visitors to the headquarters building.

Southern Lightweight Aggregate announces executive appointments

John W. Roberts, president of Southern Lightweight Aggregate Corp., Richmond, Va., has been given the additional duties of treasurer of the company. A. Cabell Ford, director of sales, was appointed secretary and Everett S. Gray was named assistant secretary.

A. F. Old, vice-president of operations continues in that capacity with his headquarters at Richmond. R. F. Gibson is the assistant secretary of the corporation with offices at Aquadale, N. C.

All officers named hold their positions both with Southern Lightweight Aggregate Corp., the parent company, and with the subsidiary group of Solite companies operating in North Carolina, Florida, and Virginia.

Metzger elected vice-president of Calaveras Cement

Directors of Calaveras Cement Co. recently elected Grant W. Metzger vice-president in charge of production. Mr. Metzger, manager of the company's San Andreas, Calif. plant since 1953, succeeds the late E. M. Barker.

Orrin E. Weeks, who since 1951 has served Calaveras as engineer, assistant quarry superintendent, mechanical superintendent, and technical assistant to the plant manager, succeeds Mr. Metzger as plant manager.

Ideal Cement sets up scholarship

Ideal Cement Co., Denver, has established a $2500 graduate fellowship in civil engineering at the University of New Mexico.

John W. Hand, business research director for Ideal, states the fellowship has been founded with a twofold purpose; first, to encourage outstanding students to further their education and secondly, to promote advanced research in the field of concrete, its components and its uses, both actual and potential.

Highlights of Vermiculite Institute annual meeting

Reports on technical advances and new market opportunities for vermiculite products highlighted the 18th annual meeting of Vermiculite Institute of Chicago, held April 4-9 at Boca Raton, Fla.

W. V. Culber, secretary-manager of Vermiculite-Northwest, Inc., Seattle, was elected president of the institute to succeed J. Brooks Robinson of Great Falls, Mont. Edward R. Murphy and Walter J. Bein were returned as managing director and treasurer, respectively.

Of special interest to the concrete industry is a new product reported at the meeting: vermiculite masonry fill, the result of 10 years of research. The fill is made water-repellent by a patented spraying process that coats the granules with a waterproofing compound. Like untreated vermiculite, it flows freely from the bag and can be quickly installed by pouring directly into cavities or the cores of blocks.

The product has passed tests developed by the National Bureau of Standards in BMS Report 82. A masonry cavity wall, deliberately built to leak, was filled with 2½ in. of water-repellent vermiculite. For 6 days (144 hr) from 5½ to 7½ in. of water was driven against the wall by a 50 to 75 mph wind. During the entire period, no water permeated across the cavity space through the vermiculite. The tests were conducted by the

ACI *Proceedings* on microfilm

ACI members and JOURNAL subscribers may obtain complete *Proceedings* volumes (including discussion) on microfilm. This is an economical means of maintaining a file of ACI material with the minimum storage space. *Proceedings* V. 46 (Sept. 1949 to June 1950) through *Proceedings* V. 54 (July 1957 to June 1958) are available.

Proceedings for V. 51 are available at $4.40; V. 52, $4.25; V. 53, $5.65; and V. 54, $5.00. For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

Structural Clay Products Research Foundation, and the fill has been approved by SCPI.

The market for vermiculite acoustical plaster in prestressed concrete construction was discussed by C. A. Pratt, vice-president of Western Mineral Products Co., Minneapolis, Minn.

Lone Star Cement sales reach new high in 1958

The 40th annual report of the Lone Star Cement Corp. shows a 5 percent sales increase to a new high of $99,788,259, representing the largest number of shipments and the highest dollar volume in its history.

When Lone Star was organized in 1919 it had an annual productive capacity of 2.8 million bbl. Today, after completion of a vast expansion program, the firm has a total capacity of 49.1 million bbl, 37.1 million of which is located in the United States. Lone Star's 15 domestic plants serve most of the nation, and its six foreign operations serve the major cement markets in Cuba, Brazil, Argentina, and Uruguay.

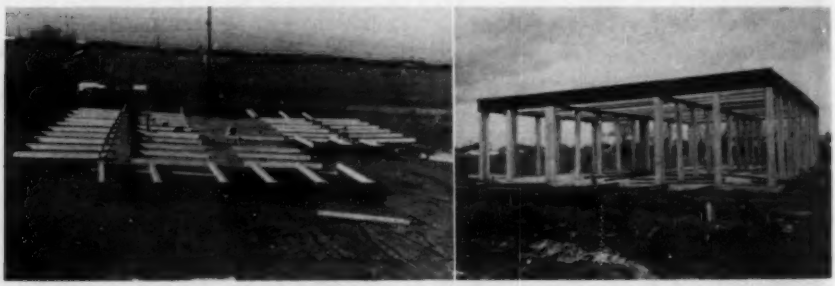
Rogers accepts post at Florida State University

Grover L. Rogers, professor of civil engineering at the Virginia Polytechnic Institute, has accepted the post of director of the Department of Engineering Science at the Florida State University in Tallahassee, effective July 1, 1959.

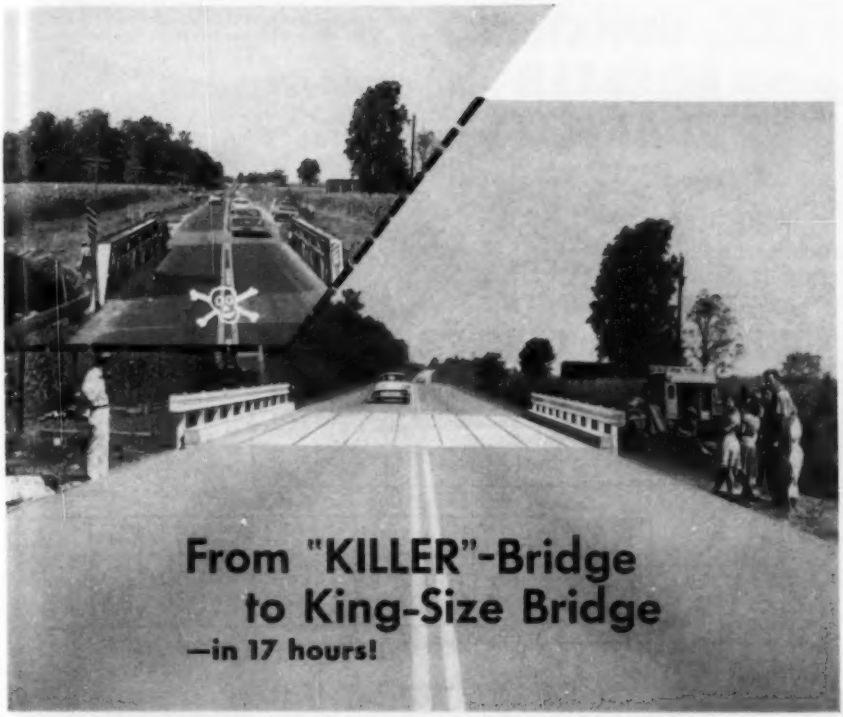
Dr. Rogers has been on the faculty of Virginia Polytechnic Institute since 1953. A graduate of VPI, he earned his advanced degrees from Harvard University, and was a Fulbright Fellow for 1 year at the University of Ghent, Belgium. He is the author of the new book, *Dynamics of Framed Structures*.

Law Engineering establishes Letcor sales division

Establishment of the Letcor Sales Division of Law Engineering Testing Co., to market diamond drill bits and sawblades, was recently announced by George H. Nelson, company president. He also announced the appointment of John H. Felton of Atlanta as manager of the new division.



EXPERIMENTAL LOW COST HOUSING—The system uses a framework of precast concrete. The T-shaped columns are about 7 x 8 in. with a 1½-in. diameter pipe protruding from top and bottom, about ½ in. on the top and 3 to 4 in. on the bottom. The bottom pipes fit into holes in the footings and slab on ground, and the top protruding pipes go through the steel channels that extend across the top of the columns. The framework is assembled on the ground and tilted into place and temporarily tied by 1-in. bars with turnbuckles. The whole operation takes about 3 hr. Precast T-joists are seated on the steel channels. The monolithic roof slab is 2 in. thick and was cast on corrugated metal sheets spanning between the joists. The interior sealing is supported on the flanges of the T-joists. Exterior and partition walls are a sandwich construction (2 in. thick for outside walls and 1½ in. thick for interior partitions) made up of insulation between two ⅛ in. cement asbestos boards. The house was designed by Alexis A. Zakharoff, architect, Chicago.



with AMDEK Concrete Beams

Only 17 hours were required by the Indiana State Highway Department to eliminate a deadly old "Killer" Bridge and replace it with a modern, wide structure near Scottsburg, Indiana. This new Amdek Bridge is not only safer to cross but is constructed of maintenance-free concrete.

American-Marietta Company makes precast concrete beams for spans up to 36 feet and prestressed beams for longer spans. All are economical and made under factory-controlled conditions—delivered to the job quickly from American-Marietta plants strategically located throughout the nation.

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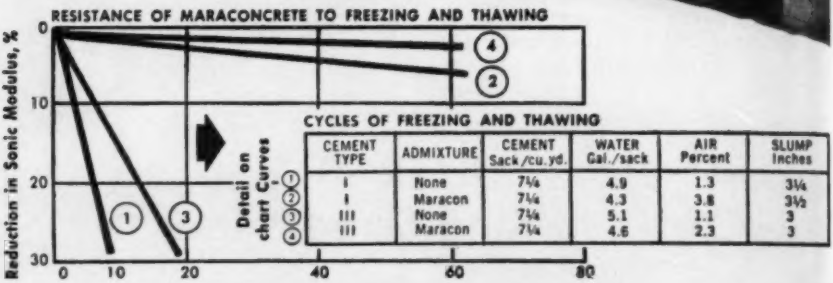

CEMENT TYPE	ADMIXTURE	CEMENT Sack/cu.yd.	WATER Gal./sack	AIR Percent	SLUMP Inches
I	None	7¼	4.9	1.3	3¼
I	Maracon	7¼	4.3	3.8	3½
III	None	7¼	5.1	1.1	3
III	Maracon	7¼	4.6	2.3	3

IMPROVE CONCRETE QUALITY: –

MARACON® promotes more complete hydration of cement particles and permits a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. Low water content means greater durability and resistance to weathering.

A. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.

B. Decrease permeability.

C. Achieve greater density and higher durability factors.

LOWER CONCRETE COSTS: –

A. Maintain slump and workability at low W/C ratios.*

B. Attain higher strengths without increasing cement content of a mix.**

C. Permit economical redesign of conventional concrete mixes.

*Decrease water by as much as 20% ...
**Increase strengths by as much as 40% ... depending on the mix design.
(The above figures are not represented as maximum. Greater percentages have been achieved in both respects.)

MARACON also reduces water requirements in concrete mixes containing pozzolanic materials.

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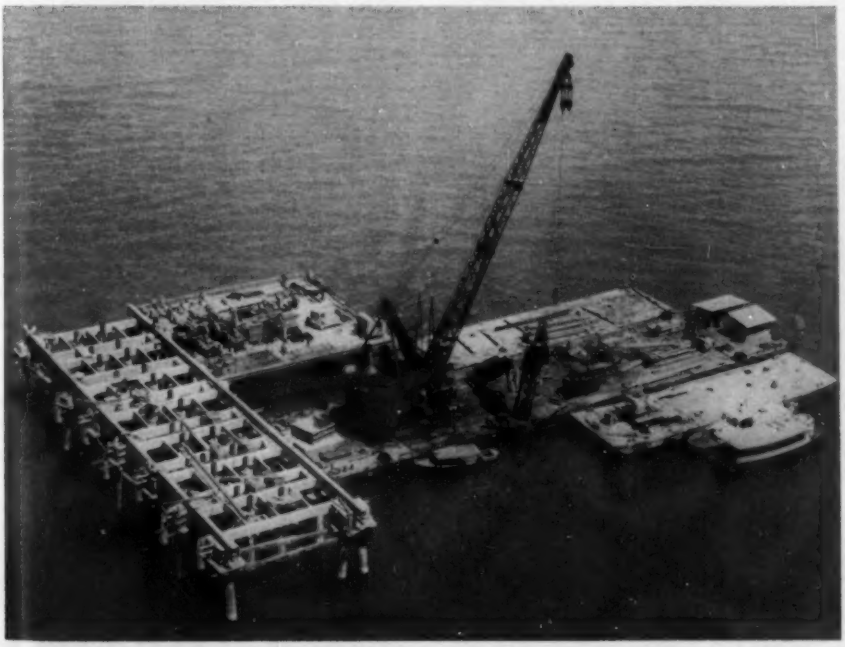
PRESTRESSED PILES

provide support for ocean oil drilling

Prestressed concrete cylinder piles 144 ft long support one of the world's largest multiple-well drilling platforms 1½ miles offshore in Trinidad's Gulf of Paria. Consisting of a 52 x 157-ft drilling deck and a 50 x 83-ft machinery deck, the 36-well platform mounts a derrick that drills six holes in one position, then skids itself ahead to drill another six holes. Maximum operational load on derrick posts is 300 tons. Each post is supported on two piles battered one to seven. Fifty of the 36-in. diameter piles were needed to support the platform, and 36 piles driven in two parallel lines will serve as conductor pipes.

Piles were manufactured in Louisiana, shipped to the site by barge, and driven by a 50-ton floating revolver crane swinging a single-acting steam hammer. Pile caps were cast in place, but deck frames were precast to speed construction. The platform's main deck consists of seven 45-ton precast frames, each 22 x 45 ft. To cut costs, six of these frames were cast in two sets of three tiers each on a barge. Lower tiers of hardened concrete served as bottom forms for each succeeding new tier as side forms were moved up.

The platform was designed and built by Raymond International, Inc.; owner is Antilles Petroleum Co., Ltd., now a part of Texaco Trinidad, Inc. Oil drilling was under way less than 6 months after the first pile was put in place.



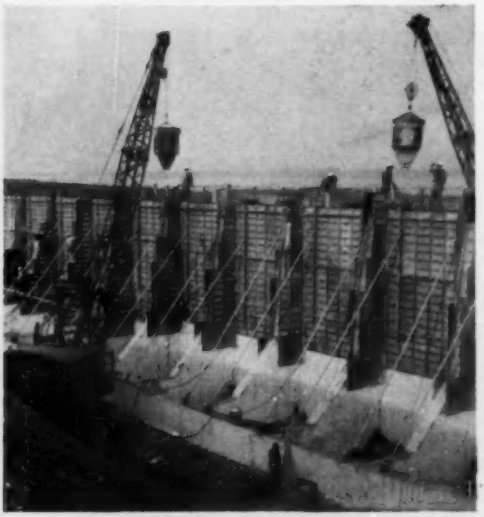
CONCRETE PERFORMANCE REPORT:

POZZOLITH concrete employed in new Air Force Academy to meet full range of engineering requirements for all types of concrete specified

Largest single construction project in U. S. Air Force history, the U. S. Air Force Academy at Colorado Springs marks a milestone in modern concrete design and construction. The project included the placing of some 800,000 cubic yards of concrete for buildings, retaining walls and bridges.

On-site concrete control lab—The Air Force Academy Construction Agency and the architects—Skidmore, Owings and Merrill—jointly supervised all construction and established an on-site concrete materials control laboratory early in 1956. During July and August, 1956—with only a few

AIR FORCE



CONCRETE RETAINING WALLS reach 36 feet high over much of the 10,000 foot wall length. Tallest pours were made by giant overhead crane. Walls required approximately 24,000 cubic yards of POZZOLITH concrete—a placeable mix of 2" to 4" slump with design strength of 3,000 psi. Contractor: T. F. Scholes, Inc., Reading, Pennsylvania. Concrete Contractor: Long Construction Co., Denver.

thousand yards of concrete placed—they observed erratic and low compressive concrete strengths. The wide range and rapid changes in temperature were suspected as the cause.

Evaluation tests of concrete materials—In August, they engaged Commercial Testing Laboratories, Denver, to make comprehensive tests of the concrete materials under the range of temperatures being experienced at the job site. Their tests clearly established that POZZOLITH would provide uniform, high strength throughout the wide range of temperature changes experienced between early morning concreting at about 50°F and midday concreting at 75° to 80°F. In September 1956, POZZOLITH was first employed in concrete at the Academy. The successful performance of POZZOLITH here led engineers to investigate the use of POZZOLITH for control of other classes of concrete—including lightweight aggregate concrete, prestressed concrete and structural concrete. *As a result of this investigation,* POZZOLITH *and only* POZZOLITH *was used as the water-reducing, set-controlling admixture for the project.*

POZZOLITH and Master Builders field service—POZZOLITH was an important aid

AERIAL VIEW of nearly completed Air Force Academy. Construction under supervision of the Air Force Academy Construction Agency. Architects: Skidmore, Owings & Merrill, Chicago • Over 24 contractors were engaged in major concrete work. Ready-mixed concrete was supplied from central batch plants operated by: Concrete Materials, Inc., Kansas City • General Concrete Co., Colorado Springs • Transit Mix Concrete Co., Colorado Springs.

ACADEMY

in meeting and exceeding specification requirements in over 750,000 of the 800,000 cubic yards of concrete at the Air Force Academy. For each type of concrete specified — it provided the required batch-to-batch uniformity, most economically, for the broad range of job requirements and varied climatic conditions encountered at the site.

The Master Builders field men and the Company engineering staff worked closely with project engineers, the field control laboratory, contractors, and concrete suppliers to achieve the common goal of uniform, superior quality concrete at lowest cost-in-place.

Neither plain concrete nor concrete with any other admixture can match the results you obtain with today's POZZOLITH. The local Master Builders field man will welcome discussing your requirements. Call him in. He's at your service—backed by the Master Builders research and engineering staff—unexcelled in the field of concrete technology. Write us for complete information on POZZOLITH.

And write for your free copy of the *detailed*
"Air Force Academy Concrete Performance Report".

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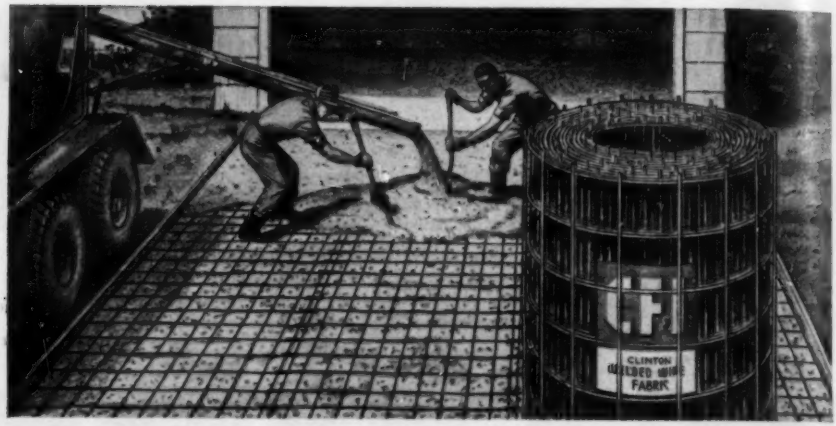

Concrete is good...

Reinforced **concrete is better**

The CF&I Image—symbol of many modern steel products for the construction industry—provides CF&I-Clinton Welded Wire Fabric for crack-resistant concrete work. A steel skeleton adds tensile strength to the concrete, allowing it to cushion the shock imposed by heavy loads. As it absorbs the stress, the steel fabric distributes it evenly in all directions. Thus, the impact is never concentrated in one area.

Steel fabric reduces the chances of cracking during setting. As it holds the concrete tightly together, it helps eliminate heaving due to extreme temperature changes. Should a tiny surface fissure develop, the steel fabric acts like a vise, preventing dirt or moisture from expanding the crack.

For complete information about economical, easy-to-install CF&I-Clinton Welded Wire Fabric, contact the nearest CF&I office.

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In the East: **WICKWIRE SPENCER STEEL DIVISION** — Atlanta • Boston • Buffalo • Chicago • Detroit • New Orleans New York • Philadelphia
CF&I OFFICE IN CANADA: Montreal • **CANADIAN REPRESENTATIVES AT:** Calgary • Edmonton • Vancouver • Winnipeg

AIA-PC Building Products Exhibition, June 23-24

The 1959 AIA-PC Building Products Exhibition will be held in conjunction with the American Institute of Architects convention in New Orleans. Most of the 75 exhibiting manufacturers have indicated that their presentations will stress design ideas to tie-in with the AIA convention theme.

Among products to be exhibited will be curtain walls with built-in air conditioning, heat pumps, and storage cabinets; curtain walls incorporating a new concept in projected type windows; and developments in concrete with emphasis on grille block curtain walls and shell roofs.

The exhibition, administered by the Producers' Council, Inc., national organization of manufacturers of building products and materials, will be at the Roosevelt Hotel.

Permanente to build plant in Hawaii

Permanente Cement Co. will build a cement manufacturing plant in Hawaii according to a recent announcement by Henry J. Kaiser, chairman of the board and president. The plant will have an initial annual capacity of 1,700,000 bbl a year.

Permanente simultaneously is expanding its Honolulu cement distribution plant by 43 percent, raising storage capacity at Iwilei Pier 32 from 80,000 to 125,000 bbl.

Superior Stone joins American-Marietta

Superior Stone Co., Raleigh, N. C., has joined the American-Marietta group of companies.

Superior Stone operates 20 quarrying and processing plants in North Carolina, South Carolina, Virginia, and Georgia. It produced 9 million tons of building materials in 1958. Reserves of slate, granite, limestone, trap rock, quartzite, and dolomite owned by Superior are estimated in excess of 800 million tons. In addition, the firm has substantial sand and gravel deposits for its long-term operations.

W. Trent Ragland, Jr., president of Superior, will continue to head the company.

Russell elected president of American Cement

Walter C. Russell was recently elected president of the American Cement Corp., Philadelphia. Mr. Russell, who is also vice-chairman of the board and chairman of the executive committee, succeeds the late Donald S. MacBride.

PCA announces executive appointments

Joseph A. Leadabrand, former manager of the Portland Cement Association's Soil-Cement Bureau, has been appointed assistant to the vice-president for promotion. He succeeds Thomas E. Long who was named eastern regional manager for the association.

E. Guy Robbins, succeeds Mr. Leadabrand as acting manager of the Soil-Cement Bureau.

North American Cement announces expansion plans

More than $1 million will be spent on expansion of the North American Cement Corp. plant at Security, Md. Ten silos, 33 ft in diameter and 140 ft high will be erected. Each of the silos will provide 20,000 bbl of storage space, more than doubling the plant's storage facilities. A modern packhouse will be erected adjacent to the silos.

Felix J. Samuely

Felix J. Samuely, consulting engineer, London, died recently.

Mr. Samuely was born in Vienna, educated in Berlin, and had been in practice as a consulting engineer in England for about 25 years. In recent years he was prominent in developing the use of precast structural members and prestressed concrete. He collaborated with architects in some notable reinforced and prestressed structures.

Among the latest buildings for which Mr. Samuely acted as structural engineer are the new United States Embassy in London; the First Presbyterian Church in Stamford, Conn., constructed entirely in precast concrete; and the British pavilions at the 1958 Brussels Universal and International Exhibition.

Fifty years of structural engineering

Institution of Structural Engineers

With the recent dedication by the American Concrete Institute of its new headquarters building, it is of interest to note that the Institution of Structural Engineers in England was celebrating its 50th anniversary at the same time. The Institution began as the Concrete Institute in 1908, founded to bring together for discussion and common action persons interested in the study and development of concrete, particularly reinforced concrete, at a time when that material was receiving scant attention from the older professional institutions.

Before the beginning

As early as the 1850's, the embedding of iron bars in portland cement concrete, to control shrinkage cracking and to supply resistance to tensile forces, was tried experimentally in France for small boats and tanks and in England for floors. This formed the basis of a structural material of some importance in the 1880's mainly through the enterprise of Hennebique and Coignet in France, of Wayss in Germany and of Ransome in the United States., but it attracted little interest in Britain. However, in 1897, an office was opened in London to exploit Hennebique's patents. A number of buildings were designed, many of which are still giving good service, and a virtual monopoly existed until Coignet, in 1904, also opened a branch in London. Within the next few years there followed a flood of patentees of deformed bars, or of systems of construction, between the merits of which the professional advisers of prospective building owners found it difficult to decide. Reinforced concrete became a kind of problem of construction, but deeply mysterious in its basis of design.

The British Fire Prevention Committee, a body of architects, surveyors, engineers, and others interested in preventing fire damage to buildings had, since 1897, been testing the fire resistance of building elements, particularly floors, including those of reinforced concrete construction. In 1906 this committee appointed a special commission on concrete aggregates and thus brought together a group of men professionally or commercially interested in reinforced concrete and aware of the paucity of reliable information as to its efficient use. Also in 1906, the Royal Institute of British Architects appointed a reinforced concrete committee to inquire into the proper conditions for the use of reinforced concrete in buildings and other structures. The War Office, the Incorporated Association of Municipal and County Engineers, the District Surveyors' Association, and the Institute of Builders collaborated and the first report of the reinforced concrete committee appeared in the RIBA Journal for June 15, 1907.

The beginning

Meanwhile, some of the professional men interested in reinforced concrete felt that if design in reinforced concrete was to be based as in other materials and disentangled from proprietary specialities, an association other than a trade association should be organized. It was decided to call the organization "The Concrete Institute."

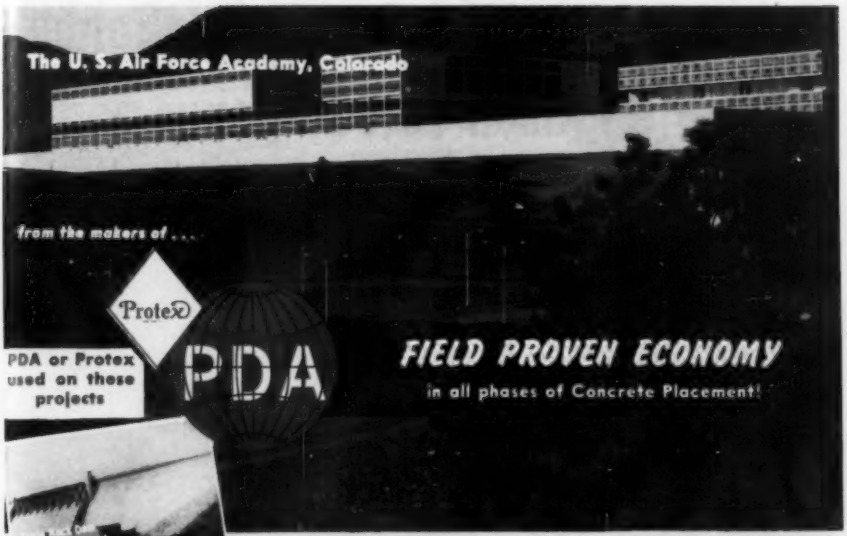
The objects of the Institute were:

(a) To advance the knowledge of concrete and reinforced concrete, and direct attention to the uses to which these materials can best be applied.

(b) To afford the means of communication between persons engaged in the design, supervision, and execution of works in which concrete and reinforced concrete are employed (excluding all questions connected with wages and trade regulation).

(c) To arrange periodical meetings for the purpose of discussing practical and scientific

Continued on p. 96



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PDA or Protex used on these projects

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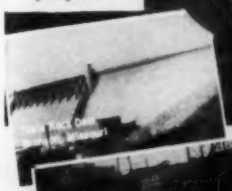
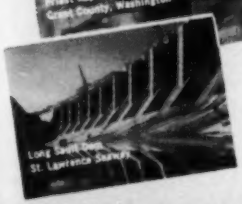
in all phases of Concrete Placement!





Among the many noteworthy and well-known projects around the world that are using PDA stands the name of the Air Force Academy in Colorado. Joining with this magnificent installation are dams, missile sites, highways, airfields, buildings . . . any construction where the efficient and economical placement of concrete is important!

PROTEX Dispersing Agent . . .

PDA increases strength at any age through water reduction and cement dispersion, improving all the desirable characteristics of concrete, giving a "live" concrete with protection against "hot" weather" slump loss and segregation — allows the placing of controlled durable air entrained concrete without plastic shrinkage cracks. PDA is a selective initial retarder— retarding only the initial set of concrete for extended vibration and finishing time (not delaying form stripping time) YET it gives no retardation in winter concreting — thus year around benefits are obtained with "all season" PDA.

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Proven in the field . . . PDA insures the successful and economical placement of better quality concrete!

Dependable — YES! Proof positive from Government and private projects. Also backed by the world-known and world-respected PROTEX name!

Adaptable to any need . . . PDA improves any Pre-stressed, Slip-form, Light-weight, Tunnelling, Tilt-up or Lift-slab concrete project!

PDA packaged in a durable bag comes to you in convenient powder form . . easily mixed and dispensed . . complete dispensing installations available upon request

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FOR BETTER AND MORE ECONOMICAL PLACEMENT OF CONCRETE, SPECIFY AND USE PDA . . . FROM THE MAKERS OF PROTEX!

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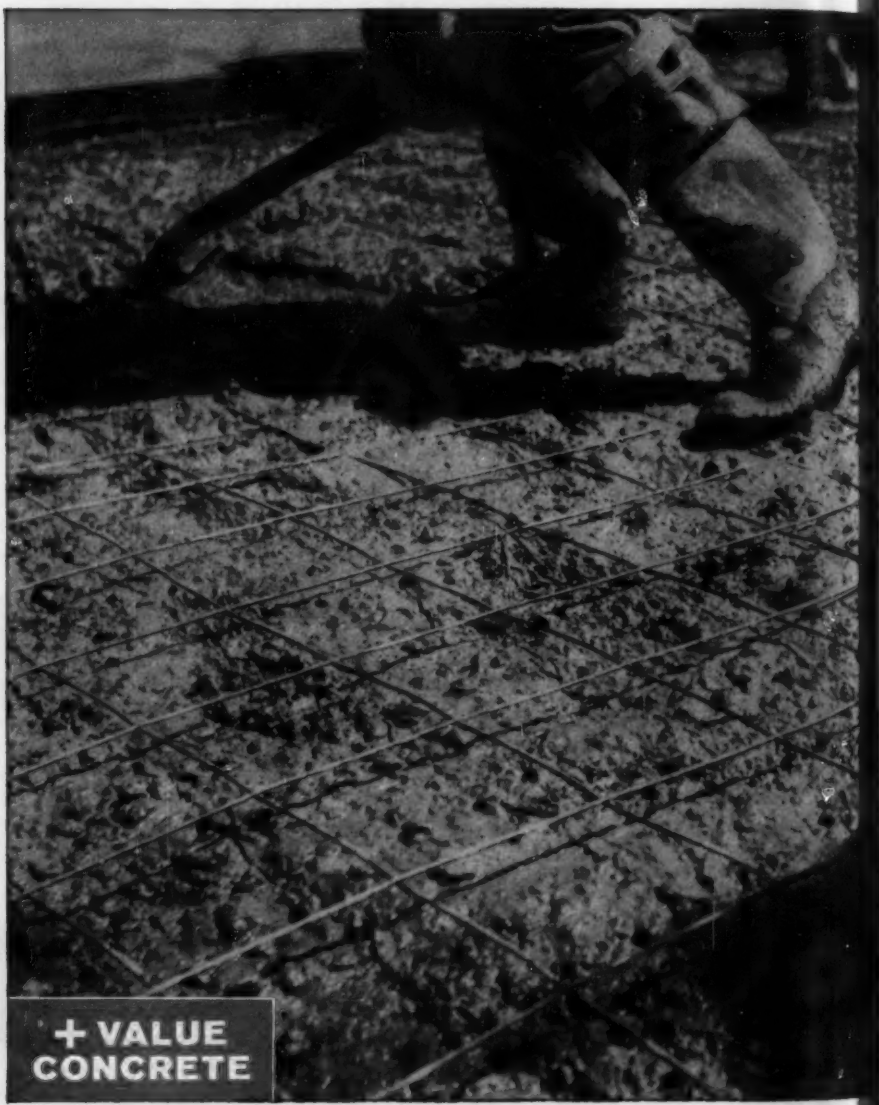
New, improved USS American Welded strength of 75,000 psi . . . and a



Wire Fabric has minimum tensile minimum yield point of 60,000 psi...

to give you a better, stronger product at no increase in cost!

For more than 50 years, USS American Welded Wire Fabric has been doing an outstanding job of reinforcing all kinds of concrete work – from porches and walks to skyscrapers and highways. And now–because of its greatly increased tensile and yield strength–it will give even greater strength, longer life, increased freedom from cracking and less maintenance. Also, it will permit longer joint spacing for reinforced slabs on ground or less steel if present joint spacing is used. The new improved Welded Wire fabric will have a 75,000 psi minimum ultimate tensile strength with a minimum yield point of 60,000 psi.

Closely controlled laboratory tests show that if the conventional bond stress theory is applied to American Welded Wire Fabric's resistance to slip, fantastically high bond stress values of from 1,000 psi to 2,700 psi are computed. (See ACI Proceedings, Vol. 48, April, 1952.) Continuing bond test research under the direction of American Iron & Steel Institute has shown such good mechanical anchorage in the concrete as to permit this increase in Tensile Strength of Fabric. American Steel & Wire is able to present this new product because of the tested bond values which enable designers to take advantage of a higher fabric yield point.

Just one example of the advantages of this improved fabric is in one-way slabs. The ACI Building Code 318-56 will allow unit tensile strength for fabric in main reinforcement of 30,000 psi in one-way slabs of 12-foot span or less, provided reinforcing members are ⅜" or less. Previously, designers were limited to 28,000 psi working stress with fabric, and only 20,000 psi with intermediate grade bars.

The new Welded Wire Fabric will cost no more. It will come in the same prefabricated rolls or sheets for easy handling and placing. Therefore, to get the improved product on your job at no extra cost, be sure to specify USS American Welded Wire Fabric.

USS American Welded Wire Fabric is available in a wide variety of styles, sizes, lengths and widths . . . in wire gauges from ½" diameter to 16 ga. and in longitudinal and transverse wire intervals of 2" to 16". Steel areas for all normal structural reinforcing in all types of construction are readily available. For more information on USS American Welded Wire Fabric–and its new tensile strength –write to American Steel & Wire, Dept. 9135, 614 Superior Avenue, N. W., Cleveland 13, Ohio.

USS and American are registered trademarks

USS **American Steel & Wire Division of United States Steel**

Columbia-Geneva Steel Division, San Francisco, Pacific Coast Distributors
Tennessee Coal & Iron Division, Fairfield, Alabama, Southern Distributors
United States Steel Export Company, Distributors Abroad

Always ask, *"is it Reinforced"*

Fifty years of structural engineering

Continued from p. 22

subjects bearing on the application of concrete and reinforced concrete, and to conduct such investigations and to issue such publications as may be deemed desirable.

Membership was open to those professionally or otherwise engaged in practical concrete work and those having some technical or literary interest in its uses.

Influence of the new group

In 1908 the London County Council proposed to amend the London Building Acts. The acts then in force made no provision at all for the permissible use of reinforced concrete and did not allow any reduction in the thickness of the external walls of a building if those walls were relieved of all load by a frame of reinforced concrete or structural steel.

The Concrete Institute, at that time, submitted its criticisms of the amendments and offered technical suggestions to be incorporated in the code. The importance of these events in the history of the Institution is that the Concrete Institute was accepted as one of the bodies that must be consulted by statutory authorities when regulations affecting the stability of buildings were in prospect.

In 1912 a proposal was made that the scope of the Institute should be widened. Steel framed buildings were increasing in number, size, and importance and none of the existing professional institutions gave them much attention. These, and other structures of all kinds, presented many problems similar to those of concrete construction and the same engineers commonly deal with more than one material. It was proposed that the Institute should cover all branches of structural engineering and the title of the organization was changed to "The Concrete Institute: an Institution for Structural Engineers, Architects, etc."

It is interesting to note that ACI at about the same time (1913) changed its name from National Association of Cement Users to American Concrete Institute.

In 1922 the name of the Institute was changed again—to the Institution of Structural Engineers. A syllabus of examinations for membership in the Institution was drawn up around 1915 (because of World War I the first candidates were not examined until 1920). In 1926 regulations were issued for the professional practice of members and a scale of charges for consulting structural engineers was adopted. In 1926 the Institution was named in the London County Council (General Powers) Act as one of the four bodies to be consulted when changes in regulations under the London Building Acts were contemplated. Also in 1926 conditions of membership were made stricter and in 1934 the royal charter was granted which guarantees the position of the Institution as the recognized national body to present the views of structural engineers on the science and art of their calling.

This brief history was abstracted from the article "The History of the Institution of Structural Engineers" in the July, 1958, jubilee issue of *The Structural Engineer*. It might be mentioned that this issue also contains a comprehensive collection of articles tracing 50 years of structural engineering.

Good Roads scholarships for Canadian students

Six scholarships for postgraduate studies in highway engineering or transportation are being awarded by the Canadian Good Roads Association for 1959. These scholarships have a total value of $13,000.

The scholarships are open to all Canadian graduate engineers and are applicable at universities in both Canada and the United States. Application forms obtainable from the association at 270 MacLaren Street, Ottawa 4, Canada.

Shilstone elected to head national laboratories group

Cecil M. Shilstone, partner, Shilstone Testing Laboratory, New Orleans, was recently elected president of the American Council of Independent Laboratories.

Ferguson joins Boeing staff

C. H. Ferguson has recently taken a position with Boeing Airplane Co., Seattle. Previously, he had been the regional manager for Intrusion-Prepakt, Inc., San Francisco.

Arkansas Cement elected to PCA membership

The Portland Cement Association has announced the election to membership of the Arkansas Cement Corp., Little Rock.

G. Donald Kennedy, PCA president, said membership was approved by the organization's board of directors and became effective March 1, 1959. The corporation, a wholly owned subsidiary of the Arkansas Louisiana Gas Co., joins more than 70 other cement manufacturers in the United States and Canada in their program to improve and extend the uses of portland cement and concrete.

Nelson appointed to Prescon sales staff

Harold L. Nelson, Albany, N. Y., has been named northeastern sales representative for The Prescon Corp., Corpus Christi, Tex.

Mr. Nelson, a graduate of Rensselaer Institute, has a background in prestressing and several years' experience as bridge engineer with the Alaska Road Commission where he was field engineer on contract bridge construction, and engaged in design of all types of steel and timber bridge structures. Before joining Prescon, Mr. Nelson was with Summers, Munninger and Molke.

Hinds elected vice-president of Master Builders

Frank R. Hinds has been elected vice-president for marketing for The Master Builders Co., Cleveland.

Mr. Hinds joined the firm in 1932 as a salesman. In 1943 he was made manager of the company's Rock Island Division and in 1957 was elected vice-president for United States sales.

Brownstead named director of Alpha Portland Cement

Ernest F. Brownstead, vice-president of operations, Alpha Portland Cement Co., Easton, Pa., has been elected to the board of directors. Mr. Brownstead, who joined Alpha in 1921, has been vice-president of operations since 1958. Prior to that he was general manager of operations.

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Bylaws Change Approved

An affirmative member vote on the proposed ACI Bylaws changes, canvassed June 1, 1959, has resulted in a dues increase. Revised membership rates will become effective July 1, 1959, as follows:

Contributing Members	$135.00
Corporation Members	65.00
Members in North America and U. S. possessions	20.00
Members elsewhere	16.00
Junior Members	10.00
Student Members	5.00

Article IV of the Institute Bylaws now reads:

Section 1. Dues shall be payable in advance on the first day of the month of notification of the Member applicant of his election by the Board of Direction, and annually thereafter, as follows: Contributing Members, $135.00; Corporation Members, $65.00; Members (individuals) in North America* and U. S. possessions, $20.00; Members elsewhere, $16.00; Junior Members, $10.00; Student Members, $5.00. Corporation Members outside North America may upon application be granted a 20 percent reduction in dues. Any individual Member may be admitted to Life Membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

*Includes all countries and territories in Central America and West Indies. Authorities: *Commercial Atlas of America*, Rand McNally & Co., Chicago; and *Survey Atlas of the World*, The Times, London.

Steinman honored by Hunter and Syracuse

At the honors convocation at Hunter College in New York City on April 29, the President's Gold Medal for distinguished service, was presented to David B. Steinman, internationally famous bridge engineer.

At commencement ceremonies at Syracuse University on June 1, the honorary degree of Doctor of Engineering was conferred on Dr. Steinman. This is his 20th honorary degree.

Patterson to manage newly organized Tretol firm

Raymond Patterson has been appointed vice-president and general manager of Tretol, Inc., Chicago, a newly formed company which will manufacture and market a line of concrete surface treatments; concrete integral mixtures; and waterproofing and dampproofing compounds.

Tretol, Inc., is an American owned and managed corporation which has arranged with Tretol, Ltd., an English firm, for the exchange and use of specific formulas and manufacturing techniques.

Stone and Webster announce promotions

M. H. Cutler has been named engineering manager of Stone and Webster Engineering Corp. with offices in New York and Boston. Mr. Cutler, formerly assistant engineering manager, succeeds Fred W. Argue who was recently named executive vice-president.

In a series of other executive shifts, Thomas A. Fearnside, chief mechanical engineer, was made assistant engineering manager. Floyd F. Dedrick, assistant chief mechanical engineer, was named to succeed Mr. Fearnside.

Texas Ready Mixed Concrete Association holds short course

Texans concerned with production, handling, and distribution of ready-mixed concrete met April 7-9 on the Texas A and M College campus for the first annual Texas Ready Mixed Concrete Association short course.

The meeting was sponsored by the Engineering Extension Service, Texas A and M College System, in cooperation with the Texas Ready Mixed Concrete Association. Programing assistance was provided by the Texas Transportation Institute and representatives of the concrete and chemical industries.

Errata

The following corrections should be made in "Pressures on Formwork," by ACI Committee 622, published in the Aug. 1958, JOURNAL.

p. 189—the equations in the middle of the page should read:

$$p = C_1\left(1 + \frac{C_2R}{T}\right)$$

and

$$p = 150\left(1 + \frac{60R}{T}\right)$$

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Honor Roll

January 1—May 31, 1959

Samuel Hobbs retains the head of the current Honor Roll having 30 credits for new members for ACI. Robert P. Witt still holds second place with 28½ credits. Being member conscious is your way of furthering the influence of the Institute and its work.

Samuel Hobbs	30
Robert P. Witt	28½
Faraj Tajirian	13
Chas. W. Cole, Jr.	9
Kenneth M. Huber	9
A. T. Klassen	9
David A. Sauer	8
George B. Southworth	8
Charles C. Luther	7
H. C. Pfannkuche	7
Henry Aaron	6
Jose Maria Bravo	6
John L. Burdick	6
S. D. Burks	6
James Chinn	6
Mauro N. Guzman	6
John E. Heer, Jr.	6
Horace G. Hill III	6
George J. Kerekes	6
George E. Large	6
W. S. Cottingham	5
Jack C. McCoe	5
Gene M. Nordby	5
Ferruh Taskin	5
Michael Alexander	4
George P. Duecy	4
Phil M. Ferguson	4
Eberhart Gunther	4
Gregorio Hernandez	4
B. E. Kester	4
S. J. Moore	4
Jerome M. Raphael	4
Joseph J. Shideler	4
C. P. Siess	4
Howard Simpson	4
James E. Stanners	4
Glenn C. Thomas	4
Lewis H. Tuthill	4
Robert F. Adams	3
Tamnoon Ansusinha	3
John D. Antrim	3
Edward E. Baker	3
Dutton Biggs	3
J. G. Bodhe	3
D. E. Bowen	3
Dan E. Branson	3
Ralph B. Brenan, Jr.	3
C. F. Brown, Jr.	3
Theodor H. Busck	3
Guillermo Castellanos G.	3
Solomon Chornik S.	3
Marvin B. Cohen	3
Ralph G. Crimms, Jr.	3
Frederick W. Drury, Jr.	3
Thomas A. Duwelius	3
R. H. Evans	3
James Lee Ford	3
Ray J. Foss	3
Russell H. Fuller	3
Junius R. Gardner	3
Ben C. Gerwick, Jr.	3
Myron L. Goral	3
Martin J. Gutzwiller	3
Robert B. Harris	3
Eric J. Hayford	3
Howard H. Hays	3
Aleck E. Hiscox	3
F. J. Hosgood	3
Samuel I. Iwata	3
J. E. Jones	3
Elias Kardaras	3
William W. Karl	3
Allan A. Kay	3
Joe W. Kelly	3
Frank Kerekes	3
Frank R. Killinger	3
Solomon Kirschen	3
William C. Krell	3
Robert R. Kuske	3
Edward Laing	3
Ronald Lazar	3
Elmer C. Lee	3
Leo M. Legatski	3
Janis Liepins	3
Henry Lipkind	3
Russ A. Loveland	3
Douglas McHenry	3
Ernst Maag	3
M. F. Macnaughton	3
Frank B. May	3
Eduardo Mercado Flores	3
William J. Moore	3
Newlin D. Morgan, Jr.	3
W. E. Moulton	3
Wendell H. Nedderman	3
Donald H. Olson	3
Rafael Enrique Pacheco	3
Orris O. Pfutzenreuter	3
Jack L. Randall	3
E. B. Rayburn, Jr.	3
Thomas J. Reading	3
Raymond C. Reese	3
Sigmund Roos	3
Antonio L. Rosquete	3
Donald A. Sawyer	3
Paul Serfass	3
L. Shector	3
Jehangir C. Shroff	3
Stuart H. Snyder	3
Ralph W. Spencer	3
W. N. Steinmann	3
Willard Stevenson	3
T. Tassios	3
F. D. Taylor	3
Pastor B. Tenchavez	3
J. Antonio Thomen	3
John P. Thompson	3
Sophus Thompson	3
Ellis S. Vieser	3
Bernardo Villegas	3
Roberto Zepeda Aldana	3
Stanley G. Zynda	3
Pedro M. Bassim	2

Walter E. Blessey 2
John E. Breen 2
Boris Bresler 2
Paul E. Chen 2
P. Q. Chin 2
Haraldo Duarte Villela 2
Ambrosio R. Flores 2
D. A. Guntin 2
Joachim F. Leppmann 2
Leo Liberthson 2
Ernest C. Marmet 2
Jorge Rodriguez Lebron 2
Emilio Rosenblueth 2
Sabri Sami 2
Nicanor B. Santos 2
K. Sitaraman 2
A. M. Tovar 2
Jose R. Perez Valentin 2
C. K. Wang 2
J. H. Weaver 2

New Members

The Board of Direction approved applications in the following catagories: 67 Individual, 1 Contributing, 1 Corporation, 11 Junior, and 34 Student, making a total of 114 new members. Considering losses due to deaths, resignations, retirements, and nonpayment of dues, the total membership on April 1, 1959 was 9897.

Individual

BETTS, FRANK A., Kansas City, Mo. (Field Engr. & Quality Concrete Engr., PCA)
BONNESEN, VIGGO, Ridgewood, N. Y. (C. E., Sigmund Roos, Cons. Engr.)
BROWN, W. P., Hawthorn, Australia (Principal, W. B. Brown & Assocs.)
BUNN, NORMAN WM., Lynnmour P. O., Canada (Gen. Mgr., Superior Concrete Products Ltd.)
BUSH, WILLIAM HEWITT, Washington, D. C. (Acting Chf. Struct. Engr., Office of Design & Constr., Public Bldgs. Service)
CARLSON, WILBUR A., Cleveland, Ohio (Tech. Dir. Matl. Testing Dept., Herron Testing Labs, Inc.)
CARSON, WALLACE J., Newport News, Va. (C. E., Tactical Air Command, USAF)
CHAPMAN, A. W., Bakersfield, Calif. (Concrete Engr., Concrete Sales, Inc.)
CHRISTODOULIDES, M. N., Nicosia, Cyprus (Exec. Engr., Pub. Wks. Dept., Cyprus Gov't)
COOK, CLYDE E., Indianapolis, Ind. (Service Engr., Ready Mixed Concrete Corp.)
COSME, LUKE, Crete, Ill. (Struct. Engr., Chicago Park Dist.)
D'ARCANGELI, FELIX ALBERTO LAZO, Santiago, Chile (C. E., Calculo Estructuras—F. Lazo)
DE LA BARRA, ALEJANDRO, Mexico, D. F., Mexico (C. E.)
DELYANNIS, LEONIDAS THEODORE, Arlington, Va. (Struct. Engr., Ben Dyer Assocs., Inc.)
DENNING, DAVID WILLIAM, Columbus, Ohio (Struct. Engr., Alden E. Stilson & Assocs., Ltd.)
DRUCKER, EMIL, Edmonton, Canada (Engrg. Student, Univ. of Alta.)
ELLESMERE-JONES, PETER D., Los Angeles, Calif. (Head, Peter D. Ellesmere-Jones & Assocs.)
FERNANDO, EUSTACE P. C., Earlysville, Va. (Civil & Struct. Engr., M. X. Wilberding)
GEORGE, CARL W., Jr., San Angelo, Tex. (Struct. Coordinator, Donald R. Goss, Arch.)
GOETTLE, RICHARD J, III, Cincinnati, Ohio (Pres. & Mgr., Richard Goettle, Inc.)
GOOD, LAMBERT E., New York, N. Y. (Constr. Rep., U. S. Corps of Engrs., Gulf Dist.)
GRANT, ALEXANDER ANDERSON, Johannesburg, S. Africa (Chf. Cost Surveyor, John Laing & Son (SA) Pty. Ltd.)
GREIG, RICHARD, Detroit, Mich. (Prod. Mgr., Water Wonderland Pools, Inc.)
GROVE, B. P., Mitcham, S. Australia (Bldg. & Civil Engrg. Contractors)
GROVE, GENE J., Estado Zulia, Venezuela (Industrial Engrg. Sect. Supv., Creole Petroleum Corp.)
GUERRERO SUAREZ, VICENTE, Lima, Peru (Chf. Chem., Compania Peruana De Cemento Portland)
HABEL, JOHN S., Sacramento, Calif. (C. E.)
HAES, ERIC F., London, England (Chm. & Mng. Dir., Sealocrete Products Ltd.)
HEIDTMANN, WALLACE H., New York, N. Y. (Pres., Purdy & Henderson Assocs., Inc.)
HENNESSEY, SILAS W., Jr., Tonawanda, N. Y. (Selling, designing, supervising construction of swimming pools)
IRANA, BEHRAM MERWAN, Bombay, India (Chf. C. E., Tata Industries Pvt., Ltd., Textile Dept.)
JACKSON, CARLTON K., Birminghan, Ala. (Assoc. Struct. Engr., Strickland & Assocs.)

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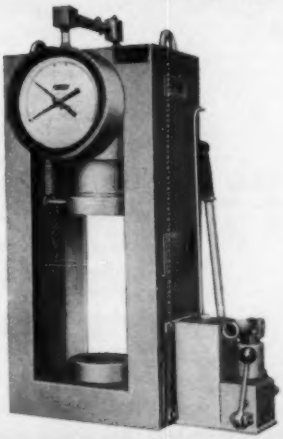

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JOHNSON, PAUL A., Youngstown, Ohio (Pres., Engrg., Adolph Johnson & Son Co.)
KESTNER, CHARLES C., Lynchburg, Va. (General Field Engr., PCA)
KOSEOGLU, H. FUAT, Houston, Tex. (Designer, Storecrafters)
LEVELIUS, WILLIAM H., Chicago, Ill. (Dist. Mgr., Pittsburgh Testing Lab.)
LOFROOS, WILLIAM NORMAN, Mt. Dora, Fl. (Interim Instructor, C. E. Dept., Univ. of Fla.)
LONG, CHARLES R., Atlanta, Ga. (Chf. Struct. Engr., Eastern Engrg. Co.)
MCCALLA, W. T., Harrisburg, Ore. (Engr., Design, Est., Morse Bros. Prestressing)
MCLAUGHLIN, CURT, New York, N. Y. (Agric. Engr., I. C. A.)
MERRIGAN, KENNETH R., Claremont, Calif. (Chf. Engr., Parker Stressed Concrete Co.)
MILLER, DANIEL J., JR., Buffalo, N. Y. (Vice-Pres. in management sales, Buffalo Crushed Stone Corp.)
MOORE, B. D., Bryan, Tex. (Plant Mgr., Bryco, Inc.)
MOSQUERA, LISANDRO, Medellin, Colombia (Construction, Ministerio de Obras Publicas)
MUSHANOW, GEORGE P., Pleasant Hill, Calif. (Struct. Designer, Bechtel Corp.)
NASH, FRANCIS C., San Carlos, Calif. (Branch Mgr., San Mateo Feed & Fuel Co.)
OFFUTT, JOE E., Oklahoma City, Okla. (Exec. Sec., Oklahoma Ready-Mixed Concrete Assn.)
PALOFF, HERBERT S., Sacramento, Calif. (Engr. "A", Aerojet-General Corp.)
PEI, I. M., New York, N. Y. (Partner, I. M. Pei & Assocs., Archs.)
POSADA URIOS, JOSE MANUEL, Mexico D. F., Mexico (C. E., Sociedad Constructora Anahuac)
PROX, FRANK A., Terre Haute, Ind. (Sales Mgr., Terre Haute Concrete Supply Corp.)
RAO, VISWANATH SUBA, Bombay, India (Res. Engr., Shalimar Tar Products (1935) Ltd.)
RAY, BERNARD JAMES, Stockport, England (Mgr., Trumix Concrete)
REEVES, HORACE A., JR., Trenton, N. J. (Proj. Engr., Atlantic Prestressed Concrete Co.)
ROBINSON, ROSS L., Quebec, Canada (Sales Mgr., Lignosol Chemicals Ltd.)
SCHOLL, JAMES L., JR., Glen Burnie, Md. (Struct. Design & Insp., Anne Arundel County Sanitary Comm.)
SCOTT, WARREN T., San Francisco, Calif. (Sales Techn., Bode Gravel Co.)
SHAVER, R. L., Commerce, Ga. (Constr. Supt., H. L. Noble Constr. Co.)
SMITH, GLEN S., Gary, Ind. (Mgr., Superior Ready Mixed Concrete Co.)
SPANNAUS, WALTER L., Duluth, Minn. (C. E., Oliver Iron Mining Co.)
STONE, W. E., Vancouver, Canada (Sales Mgr. & Chf. Engr., Western Reinforcing Steel Service Ltd.)
SWARTWOOD, GUY W., Overland Park, Kan. (Struct. Engr., Radotinsky, Meyn & Deardorff, Archs.)
TURSKI, ALVIN R., Oakland, Calif. (Design Officer, U. S. Navy Civil Engr. Corps)
WELISCH, DAVID ALAN, San Francisco, Calif. (Struct. Engr.)
WHEAT, JAMES D., Flint, Mich. (Struct. design, coordination, est., methods & materials research, A. Charles Jones, Assoc., Archs. & Engrs.)
YEE, WARREN W., Detroit, Mich. (Assoc., in charge of Struct. & Civil Engrg., Smith, Hinchman & Grylls Assocs.)
ZSCHACH, R. S., Tulsa, Okla. (Owner, Tulsa Ready-Mix Concrete Co.)

Contributing

PHOENIX CEMENT CO., Div. of American Cement Corp., Clarkdale, Ariz. (Robert F. Lowe, Chf. Chem.)

Corporation

WEBCO FORMS LIMITED, Toronto, Canada (Milton Thomas Skippon, Vice-Pres. & Mgr.)

Junior

BARACS, JANOS, Montreal, Canada (Arch. Engr., Brett, Ouellette & Blauer Assocs.)
BERGER, ALBERT J., New York, N. Y. (Sales Engr., L. Sonneborn Sons)
DRESSELHUREN, DIRK J., Montreal, Canada (Struct. Engr., Brett, Ouellette & Blauer Assoc.)
FEESER, LARRY JAMES, Boulder, Colo. (Instructor, Univ. of Colo.)
FERRE, MAURICE A., Miami, Fla. (Research Engr., Maule Industries, Inc.)
HATFIELD, HAROLD E., Abilene, Tex. (Struct. Engr., Boone & Pope)
HIGNITE, WALTER, Milwaukee, Wis. (Material Testing, Foundation Investigation, Wisconsin Testing Labs., Inc.)
HOLTZ, IGNACIO, Mexico D. F., Mexico (Struct. Designer, Diseno Racional, A. C. (Dirac)
PAPAHIU KAIKA, JUAN, Guatemala, Guatemala (C. E.)
SUGUITAN, ALBERT Q., Manila, Philippines (Proj. Insp., National Waterworks & Sewerage Authority)
VITHAL, NORI PANDURANGA, Bombay, India (Asst. Exec. Engr., Central Railway)

Student

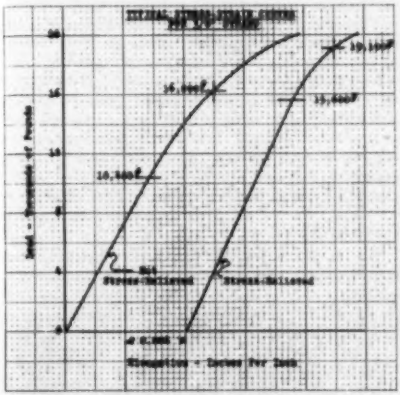
AL-YONIS, SALLEM A., Mosul, Iraq (College of Engrg.)
ATRAKCHI, MAKI A., Baghdad, Iraq (College of Engrg.)
BALIT, AMIRA, Baghdad, Iraq (College of Engrg.)
BUNNI, NAEL G., Baghdad, Iraq (College of Engrg.)
CHALOFF, CHARLES JOSEPH, Cambridge, Mass. (MIT)
CORDON M., R. O., Guatemala, Guatemala
HABBABA, M. BASHEER, Mosul, Iraq (College of Engrg.)
HADIZ, HASSOON AZIZ, Mosul, Iraq (College of Engrg.)
HADID, ZOHAIR M., Mosul, Iraq (College of Engrg.)
HAMID, IBRAHIM A., Baghdad, Iraq (College of Engrg.)
HETERICK, ROBERT C., JR., Blacksburg, Va. (Va. Polytechnic Inst.)
HUGHES, LLOYD FREDERICK, Ypsilanti, Mich. (Univ. of Mich.)
JUSTICE, JACK L., Morgantown, W. Va. (W. Va. Univ.)
KIRKLAND, CARROLL CRAWFORD, Gainsville, Fla. (Univ. of Fla.)
KO, CHIH-KUO, Minneapolis, Minn. (Univ. of Minn.)
LAMOUREUX, LUC, Sherbrooke, Canada (Univ. of Sherbrooke)
MCSWAIN, DELANO R., Atlanta, Ga. (Ga. Inst. of Tech.)
MEHTA, M. K., Bethlehem, Pa. (Lehigh Univ.)
MUHYDEN, ILHAM ABDULLA, Baghdad, Iraq (College of Engrg.)
MULET, JOSE RODRIGUEZ, Caracas, Venezuela
RADHA'A, SAMIRA M., Baghdad, Iraq (College of Engrg.)
RASSAM, MUWAFFAQ J., Mosul, Iraq (College of Engrg.)
SECOR, KENNETH E., El Cerrito, Calif. (Univ. of Calif.)
SEMPRUN BEHRENS, FRANZ, Caracas, Venezuela
SMITH, HAROLD DEAN, Cambridge, Mass. (MIT)
STOFAN S, ANTONIO, Bogota, Colombia (College of Engrg.)
STANFORD, CHARLES LEROY, Orrville, Ohio (Ohio State Univ.)
STURROCK, ROBERT CHAMBERLAIN, Wellington, New Zealand (Wellington Tech. College)
TENA GARCIA, JORGE, Mexico D. F., Mexico
VAISHNAV, RAMESHCHANDRA N., Champaign, Ill. (Univ. of Ill.)
VERNA, JOHN R., Pittsburg, Pa. (Carnegie Inst. of Tech.)
YOUNIS, ATAD MAJID, Baghdad, Iraq (College of Engrg.)
ZENNIAL, GERALD ALLEN, Brenham, Tex. (A & M College of Tex.)

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Stress-relieved strand

Bethlehem Steel Co., Inc., has developed a method of stress relieving steel strand for prestressed concrete by induction heating. With automatic control, the



process produces strand with constant and reproducible mechanical properties. The process increases the yield point of 3/8 in. diameter strand at 1 percent elongation from 16,200 to 19,100 lb. Also the proportional limit of the strand is increased form 14,000 to 15,600 lb.

The process, for which Bethlehem has filed a patent application, produces a strand that is dry and clean with no noticeable drawing lubricants on the surface of foreign material between the wires.—Bethlehem Steel Co., Inc., Bethlehem, Pa.

Form pin pulling machine

Champion Manufacturing Co. has announced the development of a concrete form pin bulling machine.

Champion's hydraulic action pin puller comes in two sizes: model with 20 in. stroke is designed to pull form pins up to and including the 24 in. size generally used on highway construction work; model with 30 in. stroke will pull 36 and 42 in. form pins generally used on airport paving jobs. The units are powered by a 5 hp Briggs-Stratton gasoline engine.—Champion Manufacturing Co., 3700 Forest Park Ave., St. Louis, Mo.

Aluminum transit mixers

Aluminum transit mixers are now being offered by Construction Machinery Co., Waterloo, Iowa.

Made of an aluminum alloy developed by Kaiser Aluminum, the Aluminum CMC Transcretes weigh 1/3 less than steel models of the same size; the heavy duty aluminum chutes weigh 60 percent less than steel chutes. They have the same basic design features as

LABYRINTH® WATERSTOPS

A SOUND INVESTMENT FOR CONCRETE CONSTRUCTION!

LABYRINTH AVAILABLE IN 2, 3 or 4 rib.

ON YOUR CONSTRUCTION:

1. Consider the investment in design, materials and labor (to mention a few).

2. Then consider how important safe, secure *watertight* concrete joints are.

3. Thorough watertightness *can* be secured by installing Labyrinth Waterstops – a dividend that makes the low initial cost of the product insignificant when compared to your total investment–and one that insures watertight concrete joints for years!

- Corrugated ribs grip concrete, insure an everlasting bond between joints.
- Finest polyvinyl plastic resists chemical action, aging, severe weather.
- Takes just seconds to nail to form . . . easy to cut and splice on location (prefabricated fittings available).
- There's a Water Seal product for every type of concrete work!

If your aim is to stop water seepage, *stop it effectively with Water Seals' Waterstops!*

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9 South Clinton Street, Chicago 6, Illinois

Made in Canada for **J. E. Goodman Sales, Ltd. Toronto, Ontario**

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Chicago 6, Illinois
Please send free sample and descriptive literature.

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Company_____

Address_____

City_____ Zone_____ State_____

CMC's complete line of steel models which are available in truck engine drive and separate engine drive models in 4, 5, 5½, 6, and 7-cu yd sizes. Complete information including illustrated booklet and specification sheet available.—Advertising and Sales Promotion Dept., Construction Machinery Co., Waterloo, Iowa

Precision bump cutter

The Concut precision bump cutter provides a practical and efficient method of eliminating bumps and rough places in concrete or asphalt surfaces according to the manufacturer. Other logical applications of the unit recommended by the company include: removing old paint markings from streets, highways, and airfields; planing bridge deckings; scarifying concrete airfield pavements, restoring skid resistant finish to worn concrete and asphalt surfaces, and bump cutting pavements before overlay.

The bumpcutter is propelled by a 70 hp air cooled engine which also drives the cutting head composed of a series of diamond blades with an over-all width of 17½ in.—Concut Sales, Inc., El Monte, Calif.

Hollow wall construction

A method of forming ducts or hollows through precast concrete floor and wall units has been patented in the United States and Canada. The method is based on two principles: first, that concrete does not readily adhere to rubber; and secondly, that rubber decreases in cross-sectional area when placed in tension.

The method uses ⅛ in. thick Linatex (or any similar elastic material) attached to wood core forms leaving an inch or so of rubber exposed at one end. When the concrete has set the Linatex is pulled out. The originator has formed a duct 25 ft long using a one-way pull and states that a duct 50 ft long can be formed by using a two-way pull. Further information available.—D. Marshall, Willroy Mines Ltd., Manitouwadge, Ontario

Jobsite batching plant

Erie Strayer's TPA-TPC portable batching plant operates either on gas or electricity and can be moved easily to keep up with progress on the job according to company spokesman.

Here are some of the features claimed for the TPA-TPC jobsite plant: set up time 3 hr; nothing is removed either for transit or operation; equipped with bases that work well in any ordinary soil; both units are mounted on rubber tires and are moved with two truck tractors.

In the TPC Automatic, both charging and discharging valves are air operated. Charging valve is equipped with electric eye for preliminary and final cutoff. Write for Bulletin TP-1.—Mitchell Advertising Agency, 3715 Poplar St., Erie, Pa.

Surface protector

A line of chemical surface protection compounds has been introduced in the United States by the Coroshield Co.

The Coroshield products are said to form a protective coating against all acids, corrosion, erosion, fouling, sea water, and weathering. It is claimed that they will adhere to all substances to which they are applied:

LOOKING AHEAD

June 21-26, 1959—Annual Meeting American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

Aug. 10-12, 1959—Conference of Lightweight Concrete Block Manufacturers, National Cinder Concrete Products Association, Chalfonte-Haddon Hall, Atlantic City, N. J.

Sept. 21-25, 1959—International Council for Building Research, Studies, and Documentation (CIB) Congress, Rotterdam, The Netherlands

Nov. 2-6, 1959—5th Annual Convention, Prestressed Concrete Institute, Deauville Hotel, Miami Beach, Fla.

Nov. 3-5, 1959—Regional Meeting, American Concrete Institute, Hotel Continental Hilton, Mexico City, Mexico

organic and inorganic; ferrous and nonferrous metals; synthetic and substitute materials including cardboard, cement, chalk, Fiberglas, glass, paper, plaster, stone, and wood.

Coroshield protective products come in liquid form and are applied by spraying, brushing, dipping, filling, or rolling. Brochure available.—Coroshield Co., Ltd., 250 West 57th St., New York 19, N. Y.

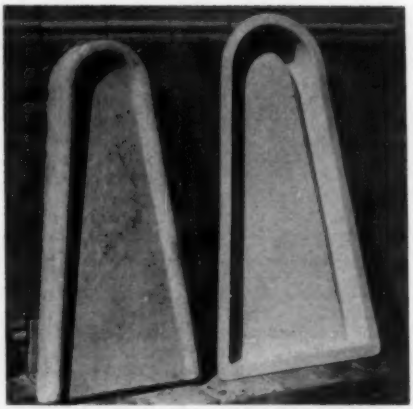
Resistant floor covering

Superior resistance to chemical attack and mechanical abuse is claimed for Emeri-Epox, a fast setting flooring material marketed by Walter Maguire Co. It is recommended for topping or resurfacing concrete, wood, tile, stone, brick, and metal flooring; for setting tile; for resistance to water, steam, oil, grease, caustics, sugars and syrups, and practically every acid.

Supplied as a package containing two components: the dry mix of emery aggregate and epoxy resin, and a liquid catalyst. These are thoroughly intermixed just prior to application on clean, oil-and grease-free surfaces. The mix is spread in place, screeded to the desired thickness, and troweled. Standard colors are gray, tile red, and tile green.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

Fiberglas forms

A line of concrete product forms, constructed of reinforced Fiberglas, has been introduced by Zeidler Concrete Products, Clear Lake, Iowa.



Zeidler, midwestern distributor of concrete block and specialty items, began using the forms for their own products 3 years ago and found the Fiberglas forms answered the need for easier stripping of precast products. Zeidler claims that some of the earlier forms are still in use after more than 300 casts, standing up well under rough usage. The forms are extremely light and resistant to acid and alkali.—Ziedler Concrete Products Co., Clear Lake, Iowa

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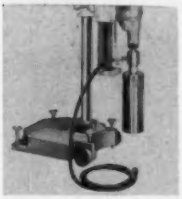
Drilling machine

The Felker Core-Lock diamond drilling machine has been designed to eliminate rough holes encountered in core drilling in reinforced concrete, masonry materials, blocks, granite, marble, or other common constructional materials according to the manufacturer.



The unit uses heavy-duty Core-Lock surface-set diamond core drills and is adjustable to feed at any angle within a full 360 deg.

Drills are used with water as a coolant, which keeps diamond rims cool and flushes the kerf of sludge. Coolant is supplied through a water swivel attached between drill and motor. Motors in several horsepower ratings and drilling speeds can be selected.—Felker Manufacturing Co., Torrance, Calif.

Sewer pipe gaskets

Tylox C-P and C sewer pipe gaskets, have been introduced by Hamilton Kent Manufacturing Co.

Tylox C-P and C gaskets are designed for concrete pipe of all sizes with single or double offset, and provide true compression, flexible, leakproof joints capable of withstanding head pressures up to 50 ft according to the manufacturer.

Made of either rubber or neoprene, gaskets consist of base, multiple sealing fins, and inspection flange which overhangs the edge of the pipe tongue, or alternately, the tongue offset, according to the type of pipe used. Illustrated engineering data, diagrams and installation information available.—A. J. Reto, sales manager, Hamilton Kent Manufacturing Co., 427 West Grand St., Kent, Ohio

Portable batching plant

Aeroil Products has introduced a 6-cu yd portable batching plant having a capacity of 40 cu yd per hr featuring Man-Ten abrasion resistant steel in the hopper.

Beam scales with over-under indicator are furnished as standard equipment. Conveyor is 24 in. wide with normal discharge height of 12 ft.

The plant is mounted for portability and is available in either gas engine or electric motor drive. Total weight

of the plant is approximately 6500 lb; over-all width, 7 ft; over-all length, 36 ft 6 in.—Aeroil Products Co., Inc., 17 Wesley St., South Hackensack, N. J.

Block unloader

The Side-O-Matic Unloader Corp. uses ductile-iron sheaves on its boom-type unloaders. The Side-O-Matic lifts and deposits groups of concrete block, brick, or palletized loads weighing up to 4500 lb. Unloading off both sides or rear, it is able to place loads over walls or other obstacles or in basement excavations to a depth of 12 ft.

Lifting and positioning of the boom is done by a hydraulic system, powered by the truck engine. The block are lifted by a six-tine fork, which is moved along the boom on a trolley. The boom swings a full 360 deg to pick up or deposit loads. The unit is factory-installed on a three-axle or four-axle truck or trailer chassis. The customer furnishes the chassis; the manufacturer supplies everything else including a specially designed heavy-duty truck bed.—T. B. Woods Sons Co., Chambersburg, Pa.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

MODIFICATION OF SOIL PROPERTIES — Detailed brochure to acquaint engineers, architects, and contractors with process by which the physical or chemical properties of a soil are altered to improve the behavior of the soil for a particular engineering purpose. Typical project reports included.—Chemject Corp., 3415-A Church St., Evanston, Ill.

MESABI SCREENS FOR RUGGED JOBS—Fully illustrated 16-page booklet describes improved lines of heavy-duty Mesabi vibrating screens for use in mines, quarries, and cement plants. Cut-a-way view of the Mesabi screen shaft assembly is included showing in detail, location of shaft, bearings, pulleys, flywheels, and other component parts.—Pioneer Engineering, Division of Poor and Co., Minneapolis 14, Minn.

ROOT-SEAL—Application and engineering data catalog sheet on Root-Seal, hot pour plastic base sewer pipe jointing compound for sealing bell and spigot sewer pipe.

GS-702—Data catalog sheet describing cold applied, ready-mix sewer joint compound, GS-702, for flexible joints on tongue and groove concrete storm and highway culvert pipe.—K. T. Snyder Co., P.O. Box 8188, 4101 San Jacinto, Houston, Tex.

PAVING MODERN CONCRETE HIGHWAYS (STF-107)—New 16-mm color film depicting base laying; form setting, subgrading; cement and aggregate batching; paver, spreader, and finisher operation; form stripping; and shoulder widening. Available to engineers, contractors, and civic groups.—Sales Promotion Dept., Construction Equipment, Blaw-Knox Co., Pittsburgh, Pa.

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All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

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Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency of government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

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LOW COST CONCRETE HIGHWAYS WITH THE SLIP-FORM PAVER—A 16-mm film, sound and color film, depicting all of the paving operations with a slip-form paver from grading and placement to the curing and sawing of joints. Twelve minutes long, the film is of special interest to engineering and contracting personnel. Available on free-loan through all Portland Cement Association district offices as well as its headquarters in Chicago.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

HOW FHA HELPS IMPROVE HOME-BUILDING TECHNIQUES—Folder explains the functions of the Federal Housing Administration and importance of FHA'S technical studies. Outlines operation and scope of the program and beneficial results to the home building industry. Ten cents per copy.—Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C.

LIGHTWEIGHT CONCRETE INFORMATION SHEET (No. 3)—Covers suggested mix proportioning for job mixed structural concrete. Prepared as a guide for proportioning concrete mixtures. Revised October, 1958.—Expanded Shale, Clay and Slate Institute, National Press Building, Washington, D. C.

WATERTIGHT CONCRETE—Factual summary of authoritative thought on the design and specification of watertight concrete is given in 6-page folder. Discussion covers the basic requirements for watertight concrete and how Pozzolith reduces permeability, shrinkage, bleeding, and segregation.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio

POWER TOOL CATALOG—Illustrated 4-page catalog describing Syntrol portable construction power tools. CATALOG NO. 591—Condensed 68-page catalog presents descriptions, data, and specifications on Syntron products. Illustrated to show products and products in operation.—Syntron Co., Homer City, Pa.

TECHNICAL BOOKS CATALOG—Lists and describes 86 technical books published by the United States Atomic Energy Commission since 1947, and 26 volumes in press or in preparation as of April 1, 1959, presenting nuclear information in readily usable form.—United States Atomic Energy Commission, Technical Information Service Extension, Oak Ridge, Tenn.

JOINTING TECHNIQUES IN STRUCTURES—Five-page booklet reprinted from *Rubber Developments*, V. 11, No. 1, Spring 1958, setting forth recent developments in jointing techniques involving the use of rubber, necessitated by the present tendency to build in large units.—Natural Rubber Bureau, 1631 K St., N. W., Washington 6, D. C.

BUILDING SAVERS FOR MAINTENANCE AND CONSTRUCTION (BP6030)—Thumbnail description of 21 Sonneborn "products" for plant maintenance and restoration. Covers products designed for floor treatment, waterproofing and dampproofing, roof coating, as well as paints and protective coatings.—L. Sonneborn Sons, Inc., Building Products Division, Department BS, 404 Fourth Ave., New York 16, N. Y.

LEAP TEES CATALOG SHEETS—Catalog section for four standard-sized Leap tees: 36 in. x 8 ft; 36 in. x 6 ft; 30 in. x 6 ft; and 24 in. x 6 ft. is now available. Catalog sheets include information on tables of loading and physical properties with a detailed sketch of each member.—Leap Associates, P.O. Box 1053, Cochrane Building, Lakeland, Fla.

CARRYING THE LOAD AT MACKINAC—New 16-mm sound film deals with construction work on the bridge with special emphasis placed on the construction of 34 prepacked concrete piers, since the superstructure phase has been covered in previous films. Available for showing before engineering societies and other technical groups.—Intrusion-Prepakt, Inc., Advertising Department, 568 Union Commerce Building, Cleveland, Ohio

Looking ahead to

1960 in New York City

ACI's Technical Activities Committee is already shaping up program plans for the 1960 convention to be held in New York City, March 14-17.

Those who intend to offer papers for presentation at the 56th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1959, and final manuscripts of accepted papers will be due Jan. 1, 1960.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1960 program

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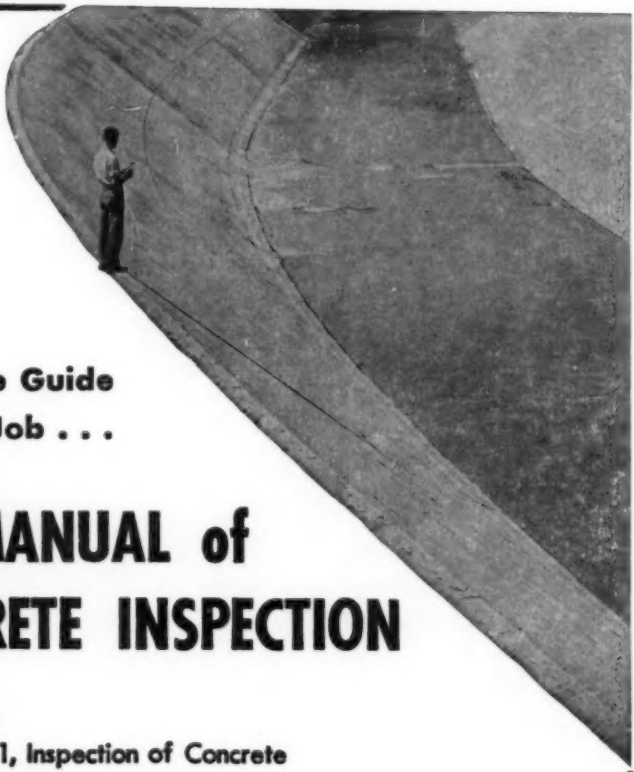

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DISCUSSION, PROCEEDINGS V. 55

Discussion of papers published in the October through December, 1958, JOURNALS appears in the concluding pages of this June issue, as the Institute continues its quarterly publication of discussion. Papers published July through September, 1958, were discussed in the March 1959 issue; discussion of papers published January through March, 1959, will be published in Part 2, September 1959 ACI JOURNAL. April through June discussion will appear in Part 2, December 1959 along with index and errata for V. 55.

Listed below are the papers whose discussion is published in this JOURNAL.

Disc. 55-4 **Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)**, ACI Committee 711..... 1331

Disc. 55-10 **Pressures on Formwork**, ACI Committee 622..... 1335

Disc. 55-32 **Shearing Strength of Prestressed Concrete Lift Slabs**, A. C. Scordelis, T. Y. Lin, and H. R. May..... 1349

Disc. 55-33 (Also 55-5, 55-16, 55-22) **Origin, Evolution, and Effects of the Air Void System in Concrete**, James E. Backstrom, Richard W. Burrows, Harry L. Flack, Richard C. Mielenz, and Vladimir E. Wolkodoff..... 1353

Disc. 55-35 **Proposed Revision of Standard 614-42: Recommended Practice for Measuring, Mixing, and Placing Concrete**, ACI Committee 614..... 1363

Disc. 55-36 **Load Factors**, International Council for Building Research (Eduardo Torroja)..... 1387

Disc. 55-37 **Moment and Shear Redistribution in Two-Span Continuous Reinforced Concrete Beams**, George C. Ernst..... 1403

Disc. 55-39 **Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings**, Jack R. Benjamin and Harry A. Williams..... 1405

Disc. 55-42 **Continuous Reinforcement in Highway Pavements**, Subcommittee VII, ACI Committee 325..... 1413

Disc. 55-43 **Stress Distribution Affects Ultimate Tensile Strength**, James S. Blackman, Gerald M. Smith, and Lyle E. Young..... 1421

Disc. 55-45 **Shear, Diagonal Tension, and Anchorage in Beams**, E. M. Rensaa..... 1427

Disc. 55-46 **Strains in Beams Having Diagonal Cracks**, D. Watstein and R. G. Mathey..... 1451

Disc. 55-47 **Influence Lines for Pressure Distribution Under a Finite Beam on Elastic Foundation**, K. C. Ray..... 1459

Discussion of a paper by A. M. Haas:

Concrete Space Structures —Relation Between Form and Structural Design*

By J. J. POLIVKA and AUTHOR

By J. J. POLIVKA†

The article is of great interest to engineers and architects and presents a good review of modern reinforced concrete space structures. This type of construction is now in general use, especially for long-span structures, and is outstanding not only for its beauty but also for economy. The author describes some typical applications, but excludes types which are not yet in general use in the United States, especially space structures in "glass-concrete." The combination of three modern structural materials—concrete, steel, and glass—has proved to be efficient for shell and dome roofs in which daylight illumination is of great importance and many European structures demonstrate this.

The writer has been interested in this type for many years, and some of his structures have been described elsewhere‡—most of them in his French book *Le Béton Translucide* which is devoted also to their structural analysis and testing.

The writer believes that it would be of interest to illustrate some of his structures of this type.

Fig. A is an interior view of a 50-ft diameter glass-concrete dome over the exhibit hall of a pavilion at the 1957 Paris World's Fair. The over-all thickness of this shell dome is 3½ in. and it rises only 4 ft. Fig. B is the interior view of the elliptic dome of the Sports Theater in Berlin.

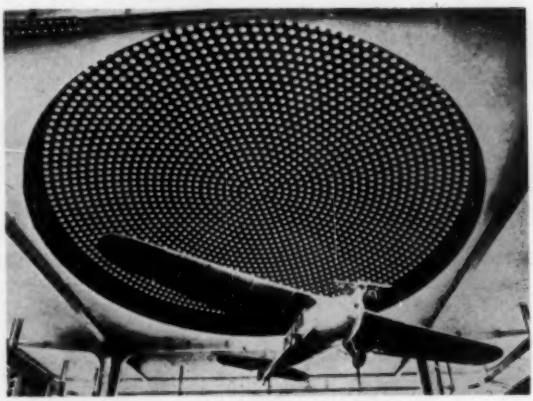


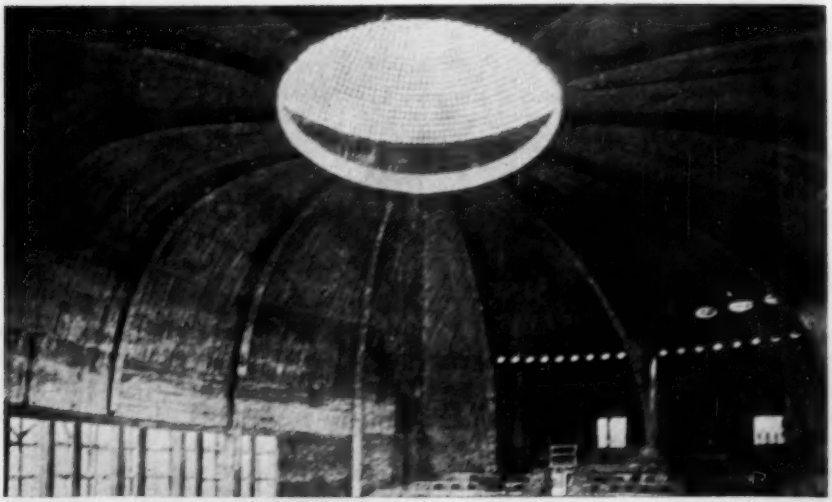
Fig. A—Exhibit hall in Paris

*ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), p. 749. Disc. 55-48 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

†Member American Concrete Institute, Consulting Engineer, Berkeley, Calif.

‡Polivka, J. J.: *Le Béton Translucide*, Etudes des Composes Siliceux, Paris and Brussels, 1938, 130 pp.; "Glass—Structural Material of Tomorrow," *Architectural Record*, Feb. 1939, pp. 66-72; "Largest 'Glass-Crete' Roof Permits High Daylight Illumination," *Architectural Record*, Oct. 1939, pp. 28-30.

There are many articles on this subject in *Czecho-Verre*, volumes for 1934-38; articles were published in English, French, German, and Czech.



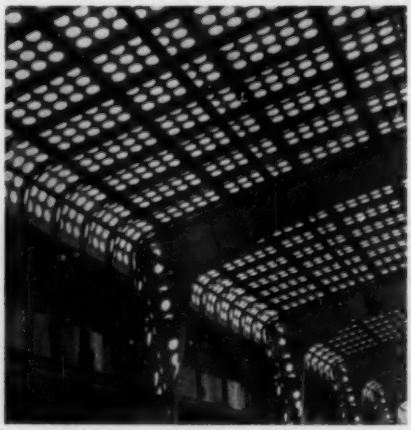


Fig. B—Elliptic dome for sports theater in Berlin with a translucent dome (above)
Fig. C—The Corn Exchange Building in Rotterdam (left)

Erected for the Olympic games in 1936, it has a maximum span of 46 ft and a gallery sidewalk around its periphery.

The Corn Exchange Building in Rotterdam (Fig. C) has several shell roofs in glass-concrete, the largest one covers 60,000 sq ft of the assembly and trading hall. Here precast shell panels in glass-concrete were used.

AUTHOR'S CLOSURE

In his discussion, Dr. Polivka points to a type of space structure which has not been included in the article, *viz.* the combination of reinforced concrete and glass. He even goes as far as classifying glass as a structural material. Although essentially he is right, the examples mentioned refer to small spans only, and show that glass behaves well under compression. As we have a fair knowledge of the proportions of glass, it may be a matter of reasoning and research to find out its behavior under tensile forces.

The article is indeed rather short and presents only an abbreviation of what, in my lectures at various universities, could be exposed more extensively.

Discussion of a paper by Edward A. Abdun-Nur and Lewis H. Tuthill:

Critera for Modern Specifications and Control*

By BAILEY TREMPER and AUTHORS

By BAILEY TREMPER†

The authors have presented important concepts that should be applied in the preparation of specifications for concrete construction and the writer is in complete agreement with the majority of them.

Item No. 7 of the criteria used on the Northern Illinois Toll Highway appears to warrant further discussion. The statement reads: "Use end-product provisions that permit maximum ingenuity to be used by the contractor to attain desired results." In the opinion of the writer, this type of provision is workable only when the quality of the material or construction can be evaluated promptly by simple tests. Unless test data are available for rendering prompt decisions at frequent intervals as to the acceptability of the work, there can be no assurance that the desired end results will be attained. The authors appear to have recognized this difficulty in their requirement for finish screening of aggregates at the batch plant. This provision requires that aggregates be handled by a particular method rather than leaving it to the contractor to produce uniform batch-to-batch gradation by whatever method he might devise. The "method" provision appears to be warranted in this case because of the obvious difficulty of making tests for grading of the batched aggregates at a pace fast enough to be of use in rejecting individual batches that do not conform.

The use of a provision for maximum water-cement ratio without a minimum cement content can lead to attempts by the contractor to work close to the specified maximum water-cement ratio. It is difficult as a control measure, to determine water-cement ratio with high accuracy because of difficulties in measuring absorption of aggregates, sampling aggregates for contained moisture, and inaccuracies in weighing batched aggregates and cement. On many projects the work will have been completed before an adequate number of strength tests can be completed to establish the necessary cement factor.

Criterion No. 19 seems to imply that it is desirable to ask for more than it is expected to receive and thus place the engineer in a good bargaining position. The relaxation of contract provisions, on projects financed by

*ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), p. 759. Disc. 55-49 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

†Member American Concrete Institute, Supervising Materials and Research Engineer, California Division of Highways, Sacramento, Calif.

public funds at least, can lead to serious complaints on the part of unsuccessful bidders who can justly state that their bids were conditioned on the expectation that the specification requirements would be enforced.

The writer does not agree with the authors' statement to the effect that once pigmented curing compounds are applied, they need no more attention. Curing compounds are effective only as long as they are maintained as a continuous intact film. If joints are sawed in pavements before the expiration of the specified curing period, the film of curing compound is necessarily broken. Abrasion by foot and other traffic is a constant source of concern and requires prompt remedial action if curing is to be completed as intended.

The authors' statements regarding the need of close coordination between the functions of design, construction, materials, and specification writing, are recognized as valid. It is also recognized that such coordination is "difficult to achieve in large organizations where specialization is the order of the day." The writer does not agree, however, that engineering consultants can necessarily do a better job of coordination. Were a consultant to assume all of the engineering duties of a large highway department, he would of necessity, need to use a large staff of engineers. If he were able to achieve better coordination, it would be because of greater administrative ability, not because he happened to be acting in a consulting capacity.

AUTHORS' CLOSURE

The authors appreciate Mr. Tremper's thoughtful discussion and particularly his perceptive definition of the limits of end-product provisions in specifications. In theory, a specification should include only end-product requirements and many construction administrators and specification writers are so bemused with this ideal that their specifications stand to lack the vitality and practicality which Mr. Tremper has emphasized so well in his second paragraph. Aside from the specific equipment and methods which experience has taught must be required in effective specifications, consistent end-product provisions should be included to allow the contractor as much freedom of action as possible, but only if the results can be evaluated as Mr. Tremper has indicated.

A contractor who works close to the limits permitted by specifications is certain to get over the line at one time or another. If he is certain that his work will be rejected when that occurs, he will see to it that he gives himself a safe margin and will control his operations in such a manner that he will stay within the specified limits. This holds true for a maximum slump requirement as well as for any other requirements. It is simply a question of whether a specification is going to be enforced.

Most specifications have maximum slump requirements which inspectors endeavor to follow but which are often, and sometimes flagrantly, exceeded. On this basis the contractor has no interest in compliance and provides no assistance in controlling concrete production. On the other hand, with a

firm policy of enforcement at reasonable limits, the entire potential of the contractor's forces become concerned with all factors causing variable and excessive slump.

As an example, a project under supervision of one of the authors, has a positive "dumping" limit on maximum slump at a practical level above the slump desired. Accordingly, the amount of water added is left entirely to the contractor—he knowing that whenever that maximum slump is exceeded, the concrete will be rejected. The contractor is careful with his water additions, and there is no problem in maintaining slump well within the range desired.

It was not intended that Criterion No. 19 imply that it is desirable to specify more than it is expected to receive or require. The intent is to place the job in a good bargaining position, not to give anything away. Usually on the job there is something worthwhile that can be obtained for any permissible concession that may develop, but standards can rarely be raised after the contract is signed, except at inordinate extra expense.

As to the use of curing compounds, it goes without saying that some surveillance is needed to make sure that the completed compound film is not impaired during the curing period. This, however, is a simple matter since an inspector walking or driving by can see at a glance any bare spots and require that they be repaired at once. This is in contrast to other curing methods which require the continuous presence of an inspector to assure proper curing.

It was not the intent of the authors to suggest that consultants should assume or coordinate functions of highway departments as may have been inferred, but rather to suggest that in many cases they might well complement departmental work by interjecting a fresh viewpoint, uninhibited by departmental administrative problems.

The authors wish to thank Mr. Tremper for his thorough discussion as it has helped to elaborate features that may not have been made clear in an attempt to keep the paper within editorial space requirements.

Discussion of a paper by Raymond J. Schutz:

Setting Time of Concrete Controlled by the Use of Admixtures*

By M. SCHUPACK†

Referring to the last paragraph on p. 779, it is stated that for cast-in-place bridges, the concrete used in the girders can be retarded sufficiently so that the concrete in the girders remains plastic until the slab is cast thus eliminating the cold joints between the girder and the slab.

The two bridges referred to, the Gorgas Lane and Wolcott Ave. bridges, both consisting of 120-ft spans, were cast-in-place on limber steel truss falsework as shown in Fig. A and B. To avoid cracking of successive lifts of concrete, due to the considerable progressive deflections of the steel truss falsework, it was essential that a retarder be used. The retarder had to assure that the concrete, particularly in the lower portion of the girders, remained plastic until the slab was placed. This meant that upward of 8 to 10 hr of retardation was needed to avoid cracks in the web of the post-tensioned bridge. A post-tensioned structure is much more sensitive to cracking since there is practically no effective reinforcing steel in the lower portions of the members in the construction stage before prestressing.

In our experience in casting over 40 individual spans for several bridges of the 120-ft span range, we have experienced a minimum of vertical cracking of the webs. The cracks that developed have been indentified, generally, with shrinkage and have completely closed up after post-stressing.

Fig. A (left)—Gorgas Lane Bridge, Philadelphia, showing the steel falsework truss. Fig. B (right)—Wolcott Ave. Bridge, Hartford, Conn., consisting of 120-ft self-centering steel truss falsework spans

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†Member American Concrete Institute, Schupack and Zollman, Consulting Engineers, Stamford, Conn.

We have found that we could not determine the amount of retarder to use only on the basis of temperature as is recommended by various vendors. As stated in this paper, there are many conditions affecting retardation. Based on our experience, we feel that the specification calling for retardation of concrete set should specify the number of hours of retardation required. To determine the quantity of retarder admixture to use, a test should be made using the particular materials to be used on the job, with due consideration being given to temperature and general weather conditions.

The other advantage that we have found with the use of retarders was the considerable amount of water reduction. For cast-in-place bridges, we prefer a slump in the range of 2 to 4 in. Using an 8-sack mix with about 30 gal. of water, this slump can be easily attained. In fact, in starting a 260-cu yd, 8-hr placement, at 6 or 7 in the morning, the slump may be as much as 5 or 6 in. with a low w/c ratio. This is particularly desirable for good concrete placing where the tendons are closely spaced at the lower portion of the girders.

We would appreciate hearing other engineers experience with retarders so that one could better predict admixture requirements for various conditions.

Discussion of a paper by Norman W. Hanson and Paul H. Kaar:

Flexural Bond Tests of Pretensioned Prestressed Beams*

By GEORGE A. DINSMORE, GENE M. NORDBY, and AUTHORS

By GEORGE A. DINSMORE†

The authors have contributed an extensive and valuable body of data on the bond phenomenon. It is unfortunate, however, that they did not make available detailed information on the cracking of specimens, for, in the writer's opinion, anchorage is utterly dependent on the location and extent of cracks at ultimate load.

The authors contend that the embedment length should be defined as the distance from the end of the member to the section of maximum moment for all cases. This does not appear realistic. Let us consider the action of under-reinforced beams as ultimate load is approached. A crack pattern has developed falling into one of the classifications shown in Fig. A.

a. *Case of a single flexural crack*—At the failure of the beam the strand force at the cracked section is the ultimate capacity of the strand and the embedment length conforms to the authors' definition.

b. *Case of inclined crack emanating from bottom fiber*—At beam failure the strand is likely to be stressed to its ultimate at the point where the crack crosses the level of the strand. The embedment length for this case can be no greater than the distance from that section to the end of the member. Over this length sufficient bond must be developed to withstand the full strength of the strand if slip is to be prevented. It is noteworthy that the formation of inclined cracks is not dependent on a prior slip of strand.‡

c. *Case of extensive flexural cracking*—As ultimate load is approached the strand yields at the crack closest to the section of maximum moment. As additional moment is applied, the strand tension and resisting compressive force remain essentially constant, but the compressive force shifts upward increasing the internal moment arm. Simultaneously the increased moment causes the steel at adjacent cracks to yield and the process is repeated until final collapse. For this case, then, it would seem only *prudent* to assume the strand to be stressed to its ultimate at the section where the outermost crack crosses the strand and to define the embedment length as the distance from that point to the end of the member.

From the foregoing it is seen that two factors must be established if strand slip is to be prevented: (a) the embedment length required to develop the full strength of the strand or "ultimate anchorage length," and (b) the effective embedment as determined by the location of the outermost crack.

*ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), p. 783. Disc. 55-51 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

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‡Walther, R. E., and Warner, R. F., "Ultimate Strength Tests of Prestressed and Conventionally Reinforced Concrete Beams in Combined Bending and Shear," *Fritz Laboratory Report* No. 223.18, Lehigh University, Sept. 1958.

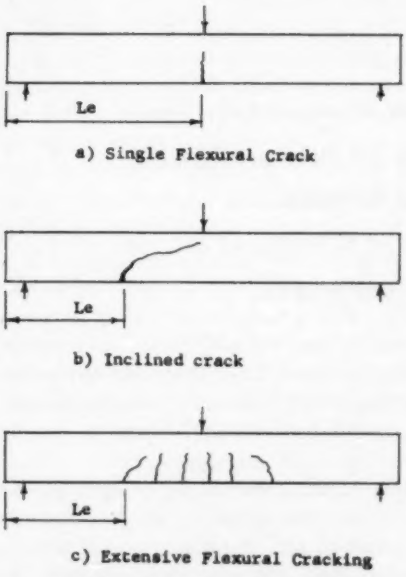

Fig. A—Classification of crack patterns

The authors determine the "critical" embedment length from the intersection of two theoretical curves in Fig. 5. Both of these curves ignore the effect of cracking. This critical length is the distance from the section of maximum moment to the end of the member required to develop the strand ultimate at that section. For ¼-in. strand this length is given as 70 in., for ⅝-in. strand as 106 in., and for ½-in. strand as 134 in. If these conclusions were valid, we would expect any underreinforced beam reinforced with ⅝-in. strand and less than 18 ft long to exhibit anchorage slip prior to failure even if only a single crack developed and that at the center line. This is quite contrary to results observed in many beam tests.* Strain data for prestressed pull-out specimens tested at Lehigh* have shown that in less than 3 ft to either side of such a crack ⅞-in. strand is fully anchored, provided that this distance does not encroach upon the transfer zone.

Thirty-four pullout specimens were tested. Columns, 4 x 4 in., were cast around a single pretensioned strand with one end of the column snug against a fixed bulkhead as shown diagramatically in Fig. B. After sudden release by burning, or gradual release by easing off on jacks at the release end, the column simulated the end block of a cracked beam. In all cases bonded lengths in excess of 48 in. developed the full tensile strength of the strand. This was also true of specimens containing strands not pretensioned. From these direct observations one concludes that an 8-ft beam reinforced with ⅞-in. strands will not show bond failure if cracking is limited to a single crack at center line.

The results of the pull-out tests were further confirmed by a study of four special beams in which bond was deliberately prohibited in the interior of the members. Bonded anchorage lengths of 48, 42, 32, and 30 in. were provided at the ends. The beam with 30 in. anchorage developed a strand force of 93 percent of rated ultimate at beam failure and showed no slip whatsoever. The specimens with 32 and 42 in. embedments showed trivial slip (0.003 in.) up to 90 percent of beam ultimate and then general slip, a continual pulling in of the strand at constant rate as the load increased. The final failure loads exceeded the computed ultimate and the strand force at failure, as determined

*Dinsmore, G. A.; Deutsch, P. L.; and Montemayor, J. L., "Anchorage and Bond in Pretensioned Prestressed Concrete Members," *Fritz Laboratory Report* No. 223.19, Lehigh University, Dec. 1958.

by strain gages, exceeded the rated strength of the strand. The specimen with 48 in. anchorage experienced a trivial slip of 0.0003 in. near ultimate load. The measured strand force in this case was the rated ultimate at beam failure. For a number of reasons developed in the report by Dinsmore, Deutsch, and Montemayor, these beam and pull-out tests are conservative with respect to the conditions obtaining in fully bonded flexural members.

The above evidence substantiates the writer's contention that the required ultimate anchorage length should be determined by test for each strand size (48 in. for 7/16-in. strand) and written into the specifications. The embedment length for a given member would then be required to exceed that value.

To find a practical means of determining embedment length, the literature was combed for tests of prestressed beams in which cracking patterns were reported. A total of 50 such tests were studied. All of the beams were reinforced by pretensioned strands of various sizes and included a wide variety of lengths, cross sections, loadings, and percentages of reinforcement. It was found that in no case had the outermost crack* developed outside a region subjected to a moment greater than 1/1.6 times the ultimate moment. Hence, for practical purposes, the location of the outermost crack is readily predicted. The applied load diagram† or curve of maximum moments is sketched with maximum ordinate taken as 1.6. The intercepts of this curve with a level line at ordinate 1.0 conservatively locates the outermost cracks. The available embedment is then evaluated. If this exceeds the ultimate anchorage length for the particular strand, the member is safe in bond.

A few additional points should be mentioned. The writer has concluded from his experience with bond tests that one of the most important factors in obtaining good bonding action is adequate vibration of the mix in the anchorage zone. In one series of tests the mix was deliberately overvibrated. The ultimate anchorage length was found to be a minimum for those specimens.

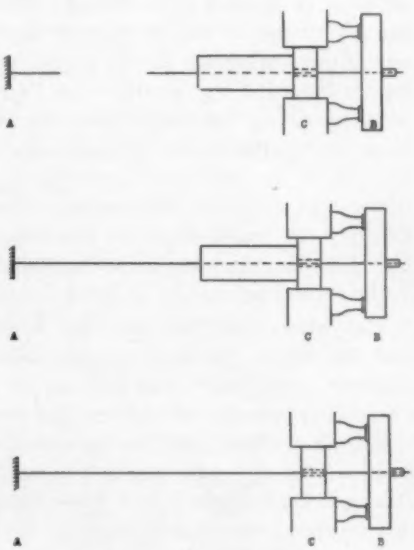

Fig. B—Bond pull-in tests

Step 1—Strand is prestressed between fixed cross-head A and movable cross-head B

Step 2—Concrete is cast around the pretensioned strand, against fixed cross-head C

Step 3—Strand is cut at end A, cross-head B is jacked away from fixed cross-head C

*Crack originating at bottom fiber. Beams with inclined cracks initiated in web were excluded from the analysis.

†Dead weight of member may usually be neglected since it is relatively insignificant for the short members in which anchorage bond may be an important consideration.

The pull-out tests indicated no significant increase in transfer length with time. This is doubtless the result of the mechanical gripping action of strands. It is generally accepted that this is not the case with smooth wires.

No consequential difference in performance was found in the pull-out tests between specimens with strand released suddenly and those gradually released. The strain distribution in the concrete as measured by strain gages on the surface was essentially the same in both cases.

The writer agrees with the authors that dynamic tests should be made, but is convinced that these will be significant only if conducted at loads in excess of those required to develop the outermost flexural cracks.

By GENE M. NORDBY*

The authors are to be congratulated on clarifying the nature of bond in strand pretensioned prestressed beams. This writer has warned engineers both formally[7,8] and orally for sometime that the slip of strand sometimes limited the usefulness of this prestressing medium. Immediately after the introduction of strand, designers treated it as a palliative for all the shortcomings of bonded prestressing. The writer does not wish to detract from the usefulness of the seven-wire strand, but only encourage engineers into seeing their materials in the proper perspective. The measurements of bond failure recorded by Nordby and Venuti[7] were often treated with scepticism because of their limited nature. The writer is grateful to Messrs. Hansen and Kaar for verifying the writer's idea of specifying "an embedment between the end of the beam and possible crack locations coupled with an allowable stress increase in the prestressing strand and the ultimate concrete strength." Entirely too much emphasis has been placed on the prestress transfer length L_t. This length is not directly related to the stress which may be developed in the strand before slip or bond failure.

The writer predicts that the ideas presented by the authors, Janney,[9] and the writer will have considerable influence on conventionally reinforced concrete. Of course, this will not be the first time that the recent intensive research in prestressed concrete has shed light on old problems. It has always baffled the writer why the so called Janney wave theory of bond has lain dormant for so many years. Actually, the theory is not a new one since Dunagen and Ernst,[10] and later Gilkey, Chamberlin, and Beal,[11] presented this concept some years ago.

The tests by the latter group were particularly significant. As part of their extensive bond tests, there was a series of pull-out specimens in which high strength alloy bars (smooth) were used. The length of the specimens varied from 3 to 24 in. and were 4 x 4 in. in cross section. The bars were ¼ and ½ in. in diameter. From these results, it was possible to construct curves similar to the authors' Fig. 5 and the writer's Fig. 17 (Reference 7). Gilkey, *et al.*, even made a prognostication as to the shape of the wave and how it

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progressed down the bar as the tension was increased. They stated "the region of maximum intensity of bond stress moves away from the loaded end as the pull increases. Between the loaded end and the region of maximum bond stress there is a somewhat uniform frictional drag resistance of greatly reduced intensity." The described paper is a classic work worth reviewing by all designers as well as researchers even though it is almost 20 years old.

Another interesting and extensive investigation on the bond of seven-wire strand was carried out at the University of Virginia.[12] The tests were too long to summarize here. It might suffice to say that the investigator invented an ingeneous method of making a prestressed pull-out specimen which nearly duplicates the action which happens in a beam. The results check the work of Messrs. Hanson and Kaar also, as far as the writer can determine.

The writer would further like to speculate on the nature of the bond failure and compare it with other phenomena in the science of the strength of materials. In doing so, the writer is trying to draw together some of the comments of the authors, the Gilkey theory, and some of his own ideas. Consider

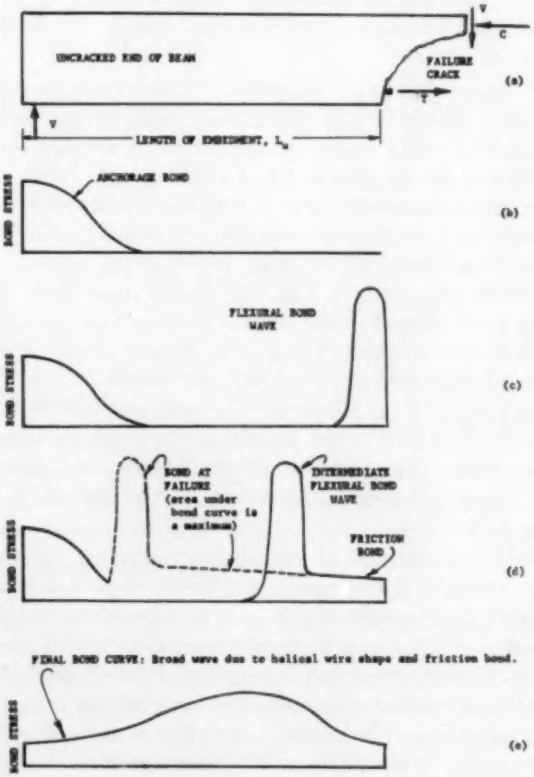

Fig. C—Bond stress curves for a prestressed beam

the uncracked end of a prestressed beam shown in Fig. C(a). Before loading, the curve of anchorage bond may be represented by the curve in Fig. C(b). The curve has been represented by the various similar theories of Janney, Guyon, and some German investigators. Most of the anchorage occurs in a short distance L_t although the bond stress is present at a low level to the center line of the beam.

After loading and development of the first crack the "wave" of failure commences. The writer thinks the wave is quite narrow and maybe does not exceed a few inches in width, but reaches bond stresses of a great magnitude (maybe several thousand psi). This is borne out by the writer's embedment length versus average bond curves in which for 9 in. embedments average bond stresses approached 1000 psi.[7] The action may be compared with the stress at the end of a crack with tension across the crack in which the theoretical infinite stress is modified to finite values by the plastic action of the material. The action might be also compared to the action of threads of a screw. As shown in photoelastic studies, the first few threads take the majority of the tension load on the bolt. The tension on a deformed reinforcing rod might have a similar action on the first lugs near the crack. The smooth bar and the deformed bar may, in addition, be similar in nature since the smooth bar is really a deformed bar in which the lug height is approaching zero.

Returning to the diagram, as the "wave" moves down the bar, the friction bond shown in Fig. C(d) takes hold adding some resistance to the pull-out force. As the wave moves further, it reaches some optimum point indicated by the dotted line in Fig. C(d). At this point the area under the curve becomes a maximum and the maximum stress is developed in the strand. As tension is increased in the strand, rapid failure and slip occurs resulting in a different bond curve made up of friction and wedging action of the helical strand as shown in Fig. C(e). The area under the curve in Fig. C(e) is not as great as that in the optimum curve in Fig. C(d) as is borne out by the drop in the developed stress at final failure of the beam. Of course, if the embedment length is great enough, the wave only partly develops or may stop at any point of progression. The only criterion would be that the area under the curve be equal to the tension in the strand. It is seen that the Gilkey theory is merely superimposed on the Janney anchorage theory.

Because of the complex nature of the failure process, it is impossible to describe it by an equation. For this reason the writer is in disagreement with the method of calculation proposed by the authors in the appendix. It seems that the calculation for u_a' is no better an approximation than that given by Eq. (10) or (11). It has always been a considerable challenge to the writer to develop instrumentation to record the shape of the bond curve. At present, the x-ray method of Evans[13] probably offers the most promise. Although some may feel that such an investigation is unduly academic and impractical, it is through such basic research which results in a real understanding of the phenomenon that a solution to the problem lies.

Along a different line of thought, it might be pointed out that slip of strand is undesirable from another point of view, even though the helical shape anchors the strand for a considerable period of loading after slip. Even small slips result in development and penetration of shear cracks. Oftentimes, this has resulted in bond failure being reported as shear or shear-moment failures. Once a slip has occurred, the mechanical shortage is not sufficient to endure until a normal flexure failure occurs. This is, of course, evident from the authors' tests.

In regard to the authors' statement about dynamic loading affecting the mechanical anchorage of the strand, the writer would like to point out the tests which have already been carried out by Nordby and Venuti.[7] In these tests dynamic loads well above the cracking point were repeated over a million times on beams with embedment lengths as low as 3 ft. To quote the paper directly: "In cases where slip was evident due to excessive static loads such slip did not progress significantly in 1,000,000 cycles when fatigue tests were continued." The data showed that the ultimate strength was not depreciated below the state the beam was in at the time it was subjected to the fatigue loading. More tests are needed, however, to completely verify this conclusion.

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AUTHORS' CLOSURE

Dr. Nordby has well illustrated the origins and again reviewed the mechanism of the bond wave theory. It is always interesting to observe how much new applications in engineering depend on principles long ago established. The aim of the authors was to apply the bond wave theory to pretensioned prestressed concrete and to produce some workable guiding criteria which can be used in structural engineering design.

The only point of disagreement between Dr. Nordby and the authors seems to be the significance of the calculation for u_a', the average bond stress between the end of the prestress transfer zone and a section of maximum strand stress. The purpose of the expression for u_a' given by Eq. (A1) is to provide an average bond value *exclusive* of the bond value over the transfer length. The authors' intention was to contribute this expression and Fig. A-1 toward better clarification of the bond picture, and to help distinguish flexural from transfer bond stress.

In reference to the discussion by Mr. Dinsmore concerning pull-out tests and beams which are partly bonded, the following comments are offered. The tests reported in our paper were of normal beams with normal percentages of reinforcement. No effort was made to simplify the beams so that analysis would be easier, because this might obscure the applicability of the test data. The disagreement between these normal flexural tests and Mr. Dinsmore's tests may lead to the conclusion that the development of bond stress in the pull-out and the modified beam tests is not the same as in a normal beam with pretensioned bonded strand. However, there may also have been a difference in the surface characteristics of the strand used in the two investigations.

A considerable part of Mr. Dinsmore's discussion centers about his assumption that, for an underreinforced beam, the steel at the location of a crack is stressed to or above the yield point, regardless of the location of the crack, when ultimate load is approached. The authors are not aware of any sound theoretical or experimental basis for such a general assumption.

During a test to ultimate strength of a prismatic beam, the initial crack will occur at the location of maximum moment. If the beam is of the end-anchored, unbonded, prestressed type, no additional cracks are apt to occur as loading continues; the initial crack will merely become more severe. If the beam is of the bonded pretensioned prestressed type, the initial crack will again occur at the point of maximum moment. When loading progresses, additional cracks on either side of this initial crack will form as bond stress develops, acting predominately in a direction toward the maximum moment section. Hence, the maximum strand stress exists at a section of maximum moment. The stress at other cracked sections equals the maximum strand stress minus the bond forces. However, for beams with variable depth, auxiliary longitudinal reinforcement or heavy concentrated loads near supports, high steel stress may develop closer to supports than the section of maximum moment. It must then be checked, by Table 4 or Fig. 6 of the paper, that such high steel stress can, in fact, be developed without slip over the available anchorage length.

For practical design purposes, it is important to note that the ultimate anchorage lengths obtained in the authors' tests are greater than those found by Mr. Dinsmore. Hence, the design procedure presented by the authors is generally conservative as compared to Mr. Dinsmore's findings.

Discussion of a report by Subcommittee VI, ACI Committee 325:

Prestressed Pavement—A World View of Its Status*

By THOMAS CHOLNOKY, L. COFF,
JOHN J. MURRAY and JOHN E. HEINZERLING, and H. KENT PRESTON

By THOMAS CHOLNOKY†

The development of prestressed concrete pavements during the last two decades has lead to some generally accepted conclusions. Design practices rather than design theories have been adopted and various construction techniques have been tried. The ground work has been completed. There is need at this time to review the past and to consider the future possibilities.

The report on prestressed pavements by Subcommittee VI, ACI Committee 325, is of great value in the first respect.

In discussing the future possibilities of prestressed concrete pavements, one should differentiate between airfield and highway pavements. The conditions of design, the possible savings in construction cost, and practical difficulties are quite different for these two applications.

It is not without reason that the largest number of major experimental prestressed pavements and also some full scale installations were built in view of or for airfield use. The reasons are obvious. The loading conditions in airfield pavements seem to suit the characteristics of prestressed concrete better. The magnitude of loads and their position in the interior of the pavement slab seem to utilize the advantages offered by prestressing more advantageously than loads applied on highway pavements. Furthermore, prestressing leads to considerable reduction of airfield pavement thicknesses whereas comparable reduction in highway pavements are not possible since they are already relatively thin. Construction problems and complicated details are less in airfield pavements where grades are generally small and without horizontal curves, and where there are few intersections or widenings.

The superior load bearing capacity of prestressed concrete has been proved repeatedly, unfortunately without sufficient conclusions as far as ultimate loads and safety is concerned. There is need to determine the minimum concrete thickness and amount of prestressing for given loads and variable base conditions. It is generally accepted that prestressed concrete pavements carry loads safely and with greater deflections than conventional

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pavements. The problem of allowable deflections in the base should therefore be re-examined to utilize the greater flexibility of prestressed concrete pavements.

Overlay pavements call for special consideration and may offer a good possibility of the application of prestressed concrete.

The two-way prestressing for airfield pavements seems to be necessary to secure uniform characteristics in the concrete sections. Highway pavements, however, do not seem to need prestressing in the transverse direction primarily because of the stresses created by the governing edge loading conditions.

The necessary amount of prestressing will be governed primarily by economy. The minimum amount to overcome friction plus 100 psi should give a sufficient strength to the pavement section to utilize the advantages of prestressing. The means of prestressing, the timing of its application, the location of the prestressing elements or tendons, if used, and the efficiency by which prestressing is developed in the pavement sections should be further studied and developed.

The full advantage of prestressed concrete sections cannot be utilized unless they are able to develop plastic hinges. Under extreme conditions such plastic hinges cannot develop when prestressing is applied by external means such as jacks. Therefore, the use of internal elements such as strands and wires seem to offer added advantages.

The application and distribution of prestressing forces is affected by the constant volumetric changes which take place in the concrete slab. It is also influenced greatly by the governing friction condition under the slab. Theories and methods should be found whereby prestressing can be developed under the most favorable conditions and with minimum losses.

The future of prestressed concrete pavements depends not only on the findings of further research and development of applicable theories—its good qualities have been already proved—but also on its cost. "Prestressed pavement is not yet competitive from a cost stand point with conventional pavements . . . " It will become so if contractors are given the chance to gain experience and perfect their techniques in conjunction with the development of improved designs and construction methods.

By L. COFF*

The report of the subcommittee would not be complete without mentioning the Chicago pavement slab of the John A. Roebling's Sons Corp., which is one of the largest experimental slabs built in this country. The slab is 144 ft long and 90 ft wide, and was built about 11 years ago.

It is correct that this is neither a road nor a runway slab, and that the load and foundation conditions are unusual because the concentration by the wooden rims of the cable reels exceeds 1000 psi as against 100 psi, a well

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known figure for rubber tires. Also, the foundation is on foundry sand of varying depth which is not quite in line with conditions of normal roads and runways. However, the principles of this slab, built in cooperation with and tested by the late C. C. Sunderland, chief bridge engineer of John A. Roebling's Sons Corp., were filed to patent in 1947 and granted to the writer in 1952. The design contains certain features for reducing subgrade friction on a jointless area of 14,000 sq ft, i.e., much larger than any of the domestic tests mentioned in the report.

The appended bibliography is so complete and in detail that a further description should not be required.

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By JOHN J. MURRAY* and JOHN E. HEINZERLING†

The subcommittee is to be complimented on its excellent and factual reporting of the work which has been done in the field of prestressed concrete pavement. This serves the useful purpose of informing the profession and should further encourage the continuation of the development into more advanced phases.

The subcommittee is also to be complimented on its brief, but pertinent analysis of the advantages of the prestressing method of pavement construction. Prestressed concrete offers to the engineer the tool needed to solve the "joint" problem. It has been recognized that the joint must inevitably be the weak point of any concrete pavement and must be the focus of eventual failure, however long delayed by refinement of treatment. To supplement the report of the subcommittee, we would like to offer the following additional comments.

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The subcommittee has pointed out, in Item 10 of the summary, that the experimental work done so far "will stimulate and benefit other investigations in their search for answers to the remaining unsolved problems." It may be emphasized here that in no other area of civil engineering is research better justified. This is because of the standardization of pavement sections used. Refinements become more essential and mistakes more costly because of repeated use of a developed design.

There is little question that prestressed concrete pavement has been demonstrated to have qualities superior to our present designs. Because of this it appears logical that refinement of design should tend toward prestressed concrete. The development of prestressed concrete pavement has reached the point where the construction should be carried into experimental traffic strips where its action under actual traffic, in various climates and various weather conditions, can be evaluated and compared with conventional pavement.

With reference to the design of a prestressed concrete traffic test pavement, we note that the subcommittee comments under "Design Practices," p. 830, that: "Design methods to date have been somewhat empirical," and further, "slab thicknesses have been selected rather than designed . . . " Although these are incontrovertible statements of fact, they are hardly to be considered departures from current practice in pavement design.

Actually all pavement slabs have been developed by field trials. It is true that analytic methods have been used to explain the action of pavement slabs and much good has come out of this work. However, the lack of a positive method of analysis to evaluate the many independent variables of subgrade condition, climate, season, thermal change rate, moisture gradient, etc., should not discourage the practical trial of prestressed concrete pavement by as wide a group of state highway departments as possible.

We are looking forward to this subcommittee continuing its good work by encouraging the initiation of test programs by highway departments supported by the Bureau of Public Roads.

By H. KENT PRESTON*

The paper gives an excellent coverage of the present status of prestressed concrete pavements.

The first sentence under the heading "Choice of Methods of Prestressing," p. 833, reads: "There are two general methods of inducing the necessary initial compressive stress in the concrete of pavement slabs: post-tensioned steel cables, and jacks reacting against abutments of some description." This statement is correct in a report dealing only with pavements that have already been built. However, such a comprehensive report seems incomplete without some mention of the pretensioned method which has recently received considerable study and offers definite advantages in many cases.

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The tensioning elements for a pretensioned pavement would be seven-wire, uncoated, stress-relieved prestressed concrete strands manufactured in accordance with ASTM A 416-57T. Long straight pavements are best adapted to the use of pretensioned strands. Briefly, the erection procedure is as follows:

1. Build abutments or drive piles at each end of the runway to serve as anchors for the initial tension in the strands until the concrete in the pavement has cured to specified strength. These abutments can easily be 12,000 to 15,000 ft apart if necessary.

2. Place seven-wire strands from anchor to anchor for the width of the first placement. If the pavement is to have transverse prestressing also, the transverse tendons will be post-tensioned. Place metal hose or other devices to make holes for the transverse tendons. Place transverse forms at expansion joints. (These joints will be 500 to 1500 ft apart depending upon friction of slab on base and details of joints being used.) Tension seven-wire strands to full load and place concrete from anchor to anchor.

3. Make second, third, etc. placements in the same manner as the first until the entire width has been placed.

4. When last placement has reached required strength, place and tension transverse tendons.

5. Release longitudinal seven-wire strands from anchorages and cut them at expansion joints.

The pretensioned bonded method has several advantages for those pavements where details permit its use.

A. Since the tendons are fully bonded, any damage is confined to the area in which it occurs.

B. Friction between the tendons and their enclosures is completely eliminated since the tendons are in the open when they are tensioned. This is important, because the distance between joints is usually long, which means large friction losses in post-tensioned tendons.

C. The designer has complete freedom in his choice of location for expansion joints. He simply places forms across the width of the pavement at points where joints are desired. When strands are cut at the anchorages they are also cut at each joint.

D. Use of seven-wire strand for the full length of the pavement reduces field labor for placing and tensioning to a minimum.

Discussion of a paper by Tung Au and Thomas D. Y. Fok:

Effects of Longitudinal Forces on Portal Frame Supporting a Highway Bridge Deck*

By H. S. GEDIZLI, KEITH A. KELLY, JASPER S. LICARI, ALEX McDONALD, DOUGLAS A. NETTLETON, and AUTHORS

By H. S. GEDIZLI†

We know that[6,7] in applying the Cross method to space frames attention must be paid to the fact that the signs of carry-over factors in bending and in torsion are opposite; i.e., after Grinter's sign convention is applied to a prismatic member, the carry-over factor in bending is $+ \frac{1}{2}$ and the carry-over factor in torsion is $- 1$.

In Table 1 of the paper all carry-over factors are considered with minus signs and no explanation is given.

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By KEITH A. KELLY‡

The authors are to be commended for developing a convenient simplification of what is normally a tedious problem in structural analysis. Unfortunately they have erred in their expression for M_{yd}. The source of this error may be traced to the failure of Eq. (3) to satisfy the requirements of static equilibrium. In its corrected form, this expression should be

$$M_{ya2} - \frac{P}{2}(c - b) = M_{yd2} + L(R_{ad2})$$

or more conveniently $M_{ya2} = - M_{yd2}$.

The corrected form of Eq. (10e) is then

$$M_{yd} = - \left[\frac{bc}{(L + 2hQ)} + \frac{2L^2 ST - bc(c - b)}{L(L + 6hQ)} \right] \frac{P}{2} - \left[\frac{LSU}{L + 6hQ} \right] M$$

*ACI JOURNAL, V. 30, No. 8, Feb. 1959 (*Proceedings* V. 55), p. 851. Disc. 55-55 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

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It may also be noted that, in their numerical example, the authors have computed the value of I_L with respect to the wrong axis and the correct expression for I_L should be

$$I_L = \frac{1}{12} \times 4 \times 3^3 = 9.0 \text{ ft}^4$$

Incorporation of these corrections in the numerical example will yield $M_{xa} = -94.7$ ft-kips, $M_{ya} = +2.1$ ft-kips, $R_{za} = +3.13$ kips, $M_{xd} = -75.3$ ft-kips, $M_{yd} = -12.8$ ft-kips, and $R_{zd} = -1.87$ kips.

Although the authors were undoubtedly primarily concerned with the solution to a problem in structural analysis, the writer believes they have lessened the value of their results by suggesting what is patently an improper application of the required loads on the portal frame.

First of all, there can be no real quarrel with the application of a longitudinal tractive force to the structure. Moving vehicles may be subject to acceleration or deceleration and such change in velocity is produced by an unbalanced force Ma. By d'Alembert's principle, this unbalanced force may be considered to act through the center of gravity of the moving vehicle. Hence the requirement that the tractive force act at a distance of 4 ft above the bridge floor. However, it is necessary to recognize the manner in which this force is transmitted to supporting pier. In most bridge structures the spans are supported at each end by fixed or expansion bearings. While both types of bearings may transmit vertical forces, all longitudinal forces are usually considered to be resisted by the fixed bearing. However, both types of bearings will usually permit rotation and are therefore incapable of transmitting any longitudinal moment. Referring to Fig. A, it is apparent that the longitudinal force of 5 kips applied 4 ft above the roadway surface is equivalent to the effect of a longitudinal force of 5 kips applied at the level of the bearing together with a vertical force equal to $5\ (10 - d)/L$. If d and L are arbitrarily assumed to be 2.2 ft and 78 ft, respectively, the resultant vertical force on the pier is 0.5 kips. It is indeed a rare structure where such a vertical load is of any significance in comparison with the dead and live load reactions. However, of more pertinence to this discussion, is the fact that the torque on the portal cap is reduced from 50 ft-kips to $5d$ ft-kips. Based on an assumed value of $d = 2.2$ ft, the reactions obtained for the authors' numerical example are: $M_{xa} = -75.4$ ft-kips, $M_{ya} = +1.0$ ft-

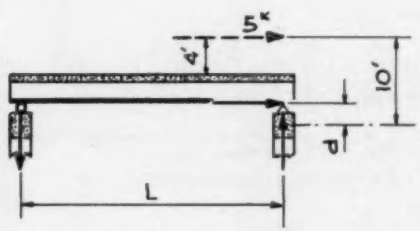
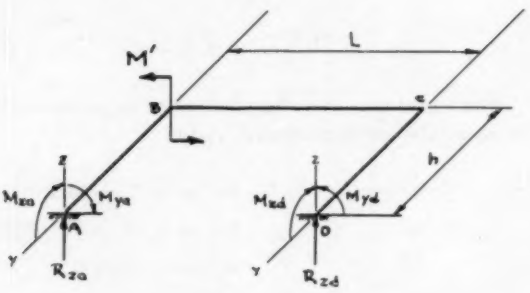


Fig. A

Fig. B



kips, $R_{za} = +3.22$ kips, $M_{zd} = -55.6$ ft-kips, $M_{yd} = -11.7$ ft-kips, and $R_{zd} = +1.78$ kips. Comparison of these results with those previously presented clearly indicates that much of the significance claimed by the authors for the effect of the tractive forces is not justified.

Furthermore, the writer assumes that the authors, by their presentation, would infer that separate tractive and frictional forces could be applied to the portal frame and the cumulative effect of such forces be determined by superposition. In so doing, however, a designer overlooks the fact that, if the longitudinal stringers support a comparatively rigid concrete roadway slab, there can be no differential longitudinal movement of the stringers. Under these circumstances, neither transverse bending in the pier cap nor torsion in the columns can exist to any significant degree. The authors' analysis is valid when the portal supports a two-girder system but is not applicable for any portal frame supporting a rigidly interconnected, multigirder system.

However the foregoing objections do not alter the fact that the authors have developed a convenient tool for the structural analyst. It is noted that the authors have not considered the effect of a longitudinal force applied to a cantilever extension of member BC. This may be solved by use of the authors' equation for a load P applied at B (i.e., $b = 0$ and $c = L$) together with a couple about the Y axis applied at B as shown in Fig. B.

Using the same notation as that presented by the authors

$$R_{za} = -R_{zd} = \left[\frac{(2h + LQ')\,Y}{hL}\right] M'$$

$$M_{xa} = -M_{xd} = -\left[\frac{(h + LQ')\,Y}{L}\right] M'$$

$$M_{ya} = \frac{M'}{L + 6hQ}\left[\frac{L\,(L + 4hQ)}{(L + 2hQ)} + 2S\,(h + 2LQ')\,Y\right]$$

$$M_{yd} = M_{ya} + R_{za}\,L - M'$$

$$\text{where } Y = \frac{6h^2Q}{|4Sh\,(h + 2LQ') + (L + 6hQ)\,(2h + LQ')|}$$

For those who may be interested in numerical results, applying these equations to the authors' frame yields

$$R_{xa} = -R_{xd} = +0.0175\,M'$$
$$M_{xa} = -M_{xd} = +0.233\,M'$$
$$M_{ya} = +0.528\,M'$$
$$M_{yd} = -0.051\,M'$$

By JASPER S. LICARI*

Messrs. Au and Fok are to be commended for their interest in laterally loaded structures as applied to highway bridges.

The procedure presented was to have overcome mathematical complexities of the general procedure for the analysis of laterally loaded plane structures, as presented by Baron and Michalos, of which a full treatment is given in the book *Theory of Structural Analysis and Design* by James Michalos.

This, however, is not in accordance with my experience with the method of Baron and Michalos. Their method involves only the use of arithmetic in a simple and well-defined computational pattern. It is ideal for design practice; more important, the method can be applied to any shape of bent, symmetrical or not, of prismatic or nonprismatic members. Furthermore, if formularization is desired the procedure by Baron and Michalos can readily be put in algebraic form for any specific problem.

The use of this method would have at least eliminated the errors in static equilibrium made by the authors in the solution of their example.

If a statical check of M_x is taken about the base of the frame it results in

$$M_{xa} + M_{xd} = 5(34) = 170 \text{ ft-kips} = (94.5 + 75.6) = 170.1 \text{ ft-kips}$$

This checks correctly. If a statical check of M_y is taken about the right leg of the frame this results in

$$R_{xa}\,(24) + M_{yd} = 5(18) - my_a \qquad R_{xa}\,(24) - 5(18) + M_{yd} + my_a = 0$$
$$3.07(24) + 28.5 = 5(18) - 4.7 \quad \text{or} \quad 73.68 - 90 + 28.5 + 4.7 = 16.88 \neq 0$$
$$102.2 \neq 85.3$$

Therefore, we do not have statical equilibrium about a y axis. The reason for this unbalance of statics is likely to be found in errors made by the authors in the mathematical formulation of the equations.

My experience has shown that there is no need to use elliptical approximations for the torsion factors because exact factors are readily available in

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Member	Length	$\frac{I}{K_{yy}}$	$\frac{I}{K_{xx}}$	dA_{yy}	dA_{xx}	X	Y	YdA_{xx}	XdA_{yy}	$Y^2dA_{xx}+i_x$	$X^2dA_{yy}+i_y$	$MX_0dA_{xx}\times10^3$	$MY_0dA_{yy}\times10^3$	$+M_xX_0\delta_{x1}$	$+M_yY_0\delta_{y1}$
AB	24	1.39	1.00	33.36	24.0	-12	12	288.0	0	4.608	4.829	-2.640	1.000	- 25.92	- 12.00
$\frac{BC}{2}$	12	1.00	1.06	12.0	12.96	-6	24	311.04	0	7.482	.576	- .324	.090	- 7.776	- .900
				45.36	36.96			599.04		12.09	5.405	-2.964	1.090	- 33.70	- 12.90
				X2 =	X2=			X2=		X2=	X2=				
				90.72	73.92			1198.1		24.18	10.81				
				A_y	A_x							P_x	P_y		

$\bar{Y}=\frac{1198.1}{73.92}=16.21'$ $\quad \nabla^2A_x=19.43 \quad 0$ $\quad \bar{X}P_y=-48.05 \quad 0$

4.75 10.81 $\quad \nabla P_x=+14.30 \quad -12.90$

$J_z=$ 15.56 $\quad M_z=$ 27.20

Calculations at "A"

$V_{zi}=\frac{M_z}{J_z}=\frac{27.20}{15.56}=1.748^k$

$M_{xi}=\left[\frac{P_x}{A_x}+\frac{M_z}{J_z}Y_0\right]=-68.44'^k$

$M_{yi}=\left[\frac{P_y}{A_y}-\frac{M_z}{J_z}X_0\right]=32.99'^k$

$V_z=\left[5.00-1.748\right]=3.252^k$

$M_x=\left[-170-(-68.44)\right]=-101.56'^k$

$M_y=\left[30-(32.99)\right]=-2.99'^k$

Fig. C

Note: Heading Xda_{yy} should read X^2da_{yy}.

Timoshenko's *Advanced Strength of Materials*, Michalos' *Theory of Structural Analysis and Design*, and other publications.

An exact solution of the illustrated problem presented by the authors is shown in Fig. C. The method is the shear and torsion analogy by Baron and Michalos. A statical and geometrical check has been made of this solution.

By ALEX McDONALD*

This is a sharp and lucid article. In swift fashion it presents the problem and gives the solution using an approach understandable to all practicing designers. Better than the guesswork too frequently resorted to, this approach aids in those complicated analyses that are often shied away from.

In the particular case of the bridge pier shown in Fig. 7, however, I believe the applied load is greater than necessary. It would seem that the longitudinal force should be transmitted through the bearings as a lateral force with no moment. While the force originates 4 ft above the floor, it has the effect, considering the bridge span as a whole, of increasing the vertical reaction at one pier and decreasing it at the other. In other words, the moment transmitted through the bearings in the numerical example should, in reality, be resisted by a vertical couple with an arm equal to the bridge span length.

This would yield a value of $5 \times 2 = 10$ ft-kips to be used in Eq. (10a) through (10f). The final reactions at the supports are then $M_{xa} = -74$ ft-kips; $M_{ya} = +3.2$ ft-kips, $R_{za} = +3.17$ kips, $M_{xd} = -56$ ft-kips, $M_{yd} = -27.5$ ft-kips, $R_{zd} = +1.83$ kips.

As a practical matter, maximum M_z and R_z are obtained with all stringers loaded. This usually leads to symmetrical loading, and consequently, a determinate structure. The other reaction, M_y, must be found by moment distribution because a maximum reaction comes from unsymmetrical loading. The value of 27.5 ft-kips causes about 35 psi torsional shear stress in the column. For the usual portal frame this stress, in combination with other shear stresses, can be tolerated without reaching critical values.

However, the more interesting point lies in the origin and delivery to the frame of an unsymmetrical, longitudinal force. While the traffic lanes can be loaded with an unbalance to one side, it would seem that an unfettered delivery of the braking force to the stringers immediately under the traffic could be accomplished only if there were no diaphragms between stringers and if there was a longitudinal joint in the slab between stringers. This idealized situation, of course, does not exist. Would it be feasible to solve the real case by analyzing the superstructure-substructure combination taking into account compatible, longitudinal strains? This article points the way to be followed concerning the substructure. Concerning the superstructure, the determination of each stringer's contribution of resistance to the braking force is not without similarities to the problem of the distribution of bending resistance in a gridwork under the action of a point load.

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It is possible, here, to cut through the maze of involvement by turning back again to the numerical example. If the superstructure tends to "even out" an eccentric, longitudinal force, there will probably be a reduction in the value that has been calculated. Yet the value used, gave only 35 psi torsional shear. Therefore, it seems reasonable to venture that shear stresses in column legs are within safe limits.

Messrs. Au and Fok are to be thanked for the solution to this problem. Without the solution there is little evidence to tell just where we stand. And certainly this is necessary for a structure which has such wide application to bridge construction.

By DOUGLAS A. NETTLETON*

The solution proposed for this problem in highway bridge design fails to take into account the interaction between the bridge deck and the supporting piers. In the usual highway bridge, there are two or more stringers supported by the portal frame, and these stringers are connected to each other by a concrete deck and diaphragms. Any tendency of the portal frame to twist about a vertical axis, due to eccentricity of the longitudinal force from one stringer, would be largely prevented by the bridge deck acting as a stiff horizontal beam between the piers at the two ends of the span. The two piers would be deflected laterally in opposite directions, but their bending stiffness in the plane of the portal would be much greater than their torsional stiffness about a vertical axis. As a result, the torsional moments in the legs would be small and the longitudinal bending moments would be nearly equally divided between the legs. There would be small bending moments in the plane of the portal frame, due to the transverse reactions induced by the deck.

The longitudinal force should be applied to the pier at its top and not 8 ft above the top as is done in the author's example. The 8-ft moment arm merely produces a longitudinal overturning effect on the span and results in vertical reactions at each end of the span, up at one end and down at the other end.

Resolution of the longitudinal force into components, as proposed for skewed piers, is a simple but probably incorrect method of taking the skew into account. This method results in a pier displacement at right angles to the bridge deck, and this displacement may be resisted by the deck, particularly in the case of a continuous bridge.

The authors seem to confine their problem to the piers supporting expansion bearings. Actually, the piers supporting fixed bearings are much more likely to receive the longitudinal force. However, if there is sufficient frictional existence in the expansion bearings to transmit the longitudinal force, there is also sufficient frictional resistance in the adjacent stringers on the same pier to transmit the restraining forces which largely prevent the twisting of the pier about a vertical axis.

*District Bridge Engineer, Texas Highway Department, Dallas, Tex.

AUTHORS' CLOSURE

The authors are indebted to the discussers for their contributions which have materially improved and clarified several points in the paper.

Since the paper deals primarily with the analysis of the structure, it has not elaborated on the transmission and distribution of the longitudinal force from the superstructure. However, the numerical example which is schematic rather than realistic has no doubt conveyed misleading implications. Thus, the amplifications on this aspect by Messrs. Kelly, McDonald, and Nettleton are most welcome. The authors agree that it is more realistic to apply the longitudinal force at the level of the bearing plus an overturning moment which causes a vertical force to the frame. The numerical example given in the paper has exaggerated the torsional effect for bridge deck with rigidly interconnected floor system.

The authors especially wish to express their sincere gratitude to Mr. Kelly for pointing out the omission of a term in Eq. (3) and tracing the subsequent error in Eq. (10e) and in the numerical example. This same error was noted by Mr. Licari who furnished an independent solution of the problem by the "shear and torsion analogy" presented by Baron and Michalos. It is regretted that such error has been made.

Mr. Licari also pointed out the generality and simplicity of the shear and torsion analogy. There is no question that it has a broader scope of application and the authors also find such methods useful. For this particular problem, however, it seems that the formulas presented in this paper involve less arithmetical operations. Whether this constitutes any time saving depends of course on the preference and experience of the designer.

Mr. Gedizli raised the question of the sign of the carry-over factors. It is a matter of convention to adopt opposite signs in bending and in torsion to facilitate the solution of complicated problems. For this problem, a different sign convention has been used consistently. As to the torsion factors used in the equations, the authors have chosen one of the many approximate expressions available and are equally willing to adopt the formulas from the sources cited by Mr. Licari. However, none of these expressions can be considered as exact when it is applied to a reinforced concrete section.

The extension of the method to include the effect of a longitudinal force applied to a cantilevered end of the pier girder by Mr. Kelly is indeed another useful addition. Mr. McDonald has kindly supplied a reasonable estimate of the torsional shearing stress in the column legs of the frame in the numerical example of the paper. If the sizes of the column legs are reduced, however, the torsional effect will become more serious.

In conclusion, the authors thank the discussers for their constructive criticisms and suggestions.

Discussion of a paper by W. C. Hansen:

Expansion and Cracking Studied in Relation to Aggregate and the Magnesia and Alkali Content of Cement*

By BRYANT MATHER and AUTHOR

By BRYANT MATHER†

Dr. Hansen's data on the possibility of interaction of two different mechanisms, alkali-aggregate reaction and delayed hydration of magnesia, both of which are well known as causes of internal expansion of hardened concrete, are valuable and useful. All of the cements studied had MgO contents below the 5.0 percent maximum noted in cement specifications, and no significant differences between the slowly cooled and rapidly cooled clinkers were detected from an examination with the petrographic microscope. It would be a valuable addition to the paper if Dr. Hansen could give any other information he may have that might bear on the question of the relative periclase and glass content, such as heats of solution, or x-ray diffraction data on the periclase contents of the slowly and quickly cooled clinkers.

Dr. Hansen's discussion of the mechanism of expansive reactions in the Kansas sand-gravel aggregate concrete is a most welcome contribution to the understanding of this phenomenon which heretofore has been regarded by many workers as inexplicable. It is particularly gratifying to observe that the suggested mechanism involves interaction of more than one factor. Feld[6] has observed with regard to structural success or failure that "usually it is a combination of conditions . . . , but not a single item by itself that can be picked as the sole and only cause of failure. Yet each in a way is what may be the responsible straw that broke the camel's back."

Dr. Hansen's experiments permit the conclusion that the two expansion-producing mechanisms under study can interact, and that such interaction could explain the expansion of the sand-gravel aggregate concrete. The author notes that his results offer no support to the suggestion that $Ca(OH)_2$ is a factor in the expansion of this concrete. Since Dr. Hansen's study is concerned with the influence of cement composition, he does not discuss the possible effects of other characteristics of the aggregate than those that may involve reaction with the cement. Rhoades and Mielenz[7] state that the

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granitic aggregates of the Platte, Republican, Kaw, and other rivers in Kansas and Nebraska, which contain high proportions of particles composed of single crystals of potassium feldspars, can cause rapid deterioration of concrete subjected to large variations in temperature, due to the difference in thermal coefficient of expansion in different directions in the feldspar crystal. They also selected gravel particles composed of individual crystals of feldspar as their illustrative example of particles having smooth, impermeable surface texture not conducive to good bond with cement.

The mechanism proposed by Dr. Hansen does not explain the performance of two of the cements used in the McPherson, Kansas, test road.[8] Sections of the test road containing Cement H, with 0.59 percent MgO, are described as severely map cracked, while sections containing Cement R, with 4.51 percent MgO, had about half as much map cracking in 1955 as the sections containing Cement H.

It is, therefore, suggested that the specific, five-step mechanism proposed by the author might more properly be regarded as *a* mechanism rather than *the* mechanism. Other combinations of phenomena that induce internal differential stress might also result in situations in which the concrete is in a locally expanded condition, from which it is partially restrained from recovery on drying (or cooling), with consequent development of cracks, which in turn not only permit access of water to periclase and also provide potential locii for deposition of secondary products which may cause further restraint during subsequent cycles of drying or cooling. I doubt that it is possible to investigate any given deteriorated concrete made with these aggregates and conclude that the deterioration was in no way significantly contributed to by any of the following:

(1) The inherently inferior paste-aggregate bond that characterizes smooth impermeable aggregate particles.
(2) The susceptibility of an inferior paste-aggregate bond to cracking due to differential expansion of an aggregate particle with widely different directional coefficients of thermal expansion when the concrete is subjected to heating and cooling.
(3) Alkali-aggregate expansion.
(4) Expansion due to delayed hydration of MgO in the cement.
(5) Restraints to shrinkage on cooling and drying due to secondary deposition in cracks; such deposits including $Ca(OH)_2$, calcium sulfoaluminate, etc.
(6) Reaction of sulfate ion with calcium aluminate in the cement.
(7) Freezing and thawing.

Dr. Hansen's conclusion suggests that for use with "the sand-gravel aggregates of Kansas, Nebraska, and Iowa" it would be prudent to require, in addition to the normal specification limits on portland cements, that they also have "alkali content of not more than about 0.6 percent and MgO content of not more than about 2.5 percent."

Dr. Hansen has kindly provided information on the autoclave expansion and the results of the chemical analysis for K_2O and Na_2O for nine of the 24 cements. These results are shown in Table A.

TABLE A—CHEMICAL ANALYSIS AND AUTOCLAVE EXPANSION OF NINE CEMENTS

Cement No.	Autoclave expansion, percent	MgO, percent	K_2O, percent	Na_2O, percent	Na_2O equivalent, percent
1Q	0.01	2.1	0.24	0.07	0.23
2Q	0.08	3.7	0.41	0.08	0.35
2U	0.11	3.5	0.31	0.08	0.28
3Q	0.14	4.9	0.26	0.07	0.24
4Q	0.06	2.2	0.42	0.51	0.79
5Q	0.08	3.6	0.42	0.51	0.79
6Q	0.17	4.0	0.51	0.38	0.71
10Q	0.08	4.6	0.26	0.08	0.25
11Q	0.15	4.7	0.33	0.54	0.76

The indicated relation of MgO content and autoclave expansion is shown in Fig. A; the correlation coefficient $r = 0.72$. The regression line indicates that an autoclave expansion of 0.05 percent or more represents cements with 2.5 percent or more MgO in this series. It is also of considerable interest to note the approximate separation of the high- and low-alkali cements. With but two exceptions (2U and 5Q), all the cements with Na_2O content greater than 0.30 percent and Na_2O equivalent content greater than 0.60 percent have higher autoclave expansions than predicted by the relation derived from all of the data, and those of lower Na_2O or Na_2O equivalent content have lower expansion than would be predicted from the regression line.

In developing an explanation of the expansion of Kansas sand-gravel aggregate concrete, it would be desirable if the explanation also accounted for the effectiveness of certain coarse aggregate additions, referred to as "sweetening," in correcting the problem. Scholer and Gibson[9] reported that "the addition of coarse aggregate, especially in amounts of 25 percent or more by weight, has been very beneficial. For the limestones it is evident that 25 percent will be adequate . . . For aggregate like the Lincoln sandstone, it appears that as much as 40 percent by weight may be required . . . Soft, absorptive sound limestones are much more effective . . . than quartzite sandstone." Farran[10] has reported data that demonstrate that portland cement paste develops bond of greater strength to clean polished calcite than to clean polished quartz because of the epitactic growth of calcium hydroxide on the calcite. Siliceous aggregates containing forms of silica capable of participating in alkali-aggregate reaction may develop superior bond with cement paste as a result of the reaction.[11] Cherts containing chalcedony are aggregates of this type.

In considering the role of MgO in producing expansion, it would be desirable to have information on the reactions in which the MgO participates and the reaction products formed. Wells, Clarke, Newman, and Bishop[12] in describing failures of white-coat plaster note that overburned MgO in dolomitic finishing lime in the wall hydrates gradually to $Mg(OH)_2$ which, in turn, may carbonate to $MgCO_3 \cdot 3H_2O$. The volume increase from MgO to $Mg(OH)_2$ is 125 percent; that from $Mg(OH)_2$ to $MgCO_3 \cdot 3H_2O$ is 205 percent, or a total of 586 percent from MgO to $MgCO_3 \cdot 3H_2O$. The product $MgCO_3 \cdot 3H_2O$ which is identical to the mineral nesquehonite has been found

as a deposit on and in concrete that had been exposed to Mississippi River water with an Mg content of about 10 ppm. If expanded sand-gravel concrete, not exposed to an outside source of MgO, could be shown to contain $Mg(OH)_2$ or $MgCO_3 \cdot 3H_2O$, it would tend to support the author's hypothesis

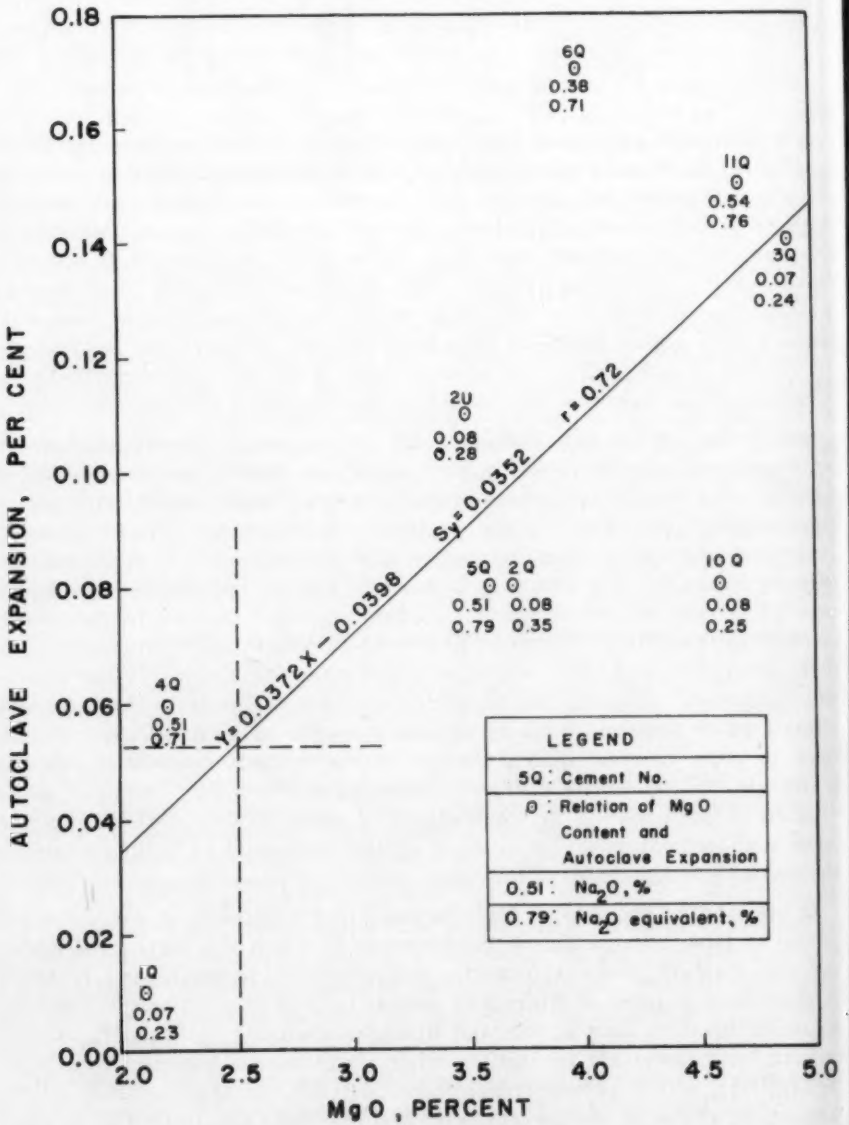

Fig. A—Relation of MgO content and autoclave expansion for nine portland cements

as to the importance of the role played by MgO and also suggest something of the quantitative relations between quantity of MgO reacted and amount of volume increase produced.

ACKNOWLEDGMENTS

The preparation of this discussion was contributed to materially by Thomas B. Kennedy, Katharine Mather, and Leonard Pepper, whose assistance is acknowledged with appreciation.

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AUTHOR'S CLOSURE

The discussion by Mr. Mather suggests a number of factors that might contribute to the deterioration of concrete. It seems that there can be no question that, once some type of deterioration has started, other types may augment it. Therefore, the problem of building durable concrete structures is one of minimizing factors that can start deterioration. Concrete technologists have learned that, by the use of air-entrained concrete in well designed mixes, they can prevent freezing and thawing from starting deterioration. However, protection against freezing and thawing does not appear to be the type of protection needed to prevent deterioration of concrete made with certain sand-gravel aggregates. The problem of the concrete technologist, when using these aggregates, appears to be one of finding the cause of the initial deterioration.

I have suggested that the cause of the initial deterioration in the case of certain sand-gravel aggregates is a cracking of the cement paste caused by the alkali-aggregate reaction which is Item 3 of the seven items listed by Mr. Mather. His Item 4 appeared to be the secondary cause in our project.

It seems that concrete structures throughout the world bear witness to the fact that what may be classed as good or reasonably good concrete from the standpoint of cement content, air content, water-cement ratio, quality

of aggregate, and quality of workmanship will withstand for long periods of time the deteriorating forces of nature. Hence, when concrete which seems to rate this classification deteriorates in a relatively short period of time, one has to search for the key that unlocked the door to one or more deteriorating forces. In this paper, I concluded that the alkali-aggregate was that key and that it operated by increasing the permeability of the cement paste to water.

Our specimens were on well drained ground in a relatively cold climate. They probably were rarely exposed to frost in a saturated condition. If they had been so exposed, a secondary deteriorating force may have been freezing and thawing and, then, the role of periclase might have been less distinct than it appeared to be.

Mr. Mather refers to the fact that, in the McPherson, Kansas, test road there does not appear to be a correlation between the MgO contents of the cements and the deterioration. The alkali contents of the cements were similar and, accordingly, if alkali-aggregate reaction was the primary cause of the deterioration, all of the concretes should have been equally susceptible to secondary deterioration. However, the concrete made with the cement of highest MgO content (Cement R) shows little, if any, deterioration; whereas that made with the cement of lowest MgO content (Cement H) is badly cracked. I have no explanation for the behaviors of these cements except to suggest that there were differences, either in the concretes or the exposures, that are not apparent from the available data. It is extremely difficult in a field project to eliminate all variables.

Mr. Mather raises the question as to why the addition of coarse aggregate to the concrete made with certain sand-gravel aggregates reduces cracking. It seems that this might be due to the coarse aggregate restraining the expansion caused by alkali-aggregate reaction, as well as the shrinkage during drying. This restraint might be such as to minimize cracking of the cement paste and, accordingly, minimize any increase in permeability from this reaction.

The nine cements from which data were used by Mr. Mather in Fig. A were the cements that were used either alone or as blends in obtaining the data of Table 3 and Fig. 1 of the paper.

Mr. Mather's discussion serves to point out that there is need for more work on concrete made with aggregates that may react with alkalies and aggregates made up of crystals in which the thermal coefficients of expansion vary in different directions. It seems that this variation in thermal coefficient could cause the paste to crack and become susceptible to attack from secondary causes.

Discussion of a paper by Shu-t'ien Li:

Wear-Resistant Concrete Construction*

By CHARLES B. ELDER, JR., A. B. FOWLER, DANIEL A. GUNTIN, and AUTHOR

By CHARLES B. ELDER JR.†

The adoption of two-course floor construction is growing measurably and reports such as Dr. Li's will be instrumental in increasing the quality of such construction. Skilled workmanship, proper materials, and the correct procedure will combine to produce economically a surface that will endure severe conditions for a long period of service with little or no maintenance.

The success of such construction begins with the specifications concerning the base slab and the wearing surface. These should be strict, easily understood by the men in the field, and *adhered* to. With respect to the wearing surface, the material costs are such a relatively small proportion of the overall costs that the best available should be specified and used.

For toppings that are to be applied after the base slab has hardened, Dr. Li suggests that "the base slab should be thoroughly wetted just prior to placing the wearing course but no pools of water left standing." This has given excellent results and has been the accepted procedure. However, in a recent paper by Earl J. Felt of the Portland Cement Association entitled "Resurfacing and Patching Concrete Pavement with Bonded Concrete," he points out that his test results agreed with Withey,[12] Thornton,[13] and Waters[14] that the bond is strongest when the surface of the base slab is dry. While this is in conjunction with studies on concrete pavement it may warrant further study with respect to floors. Mr. Felt does say that the apparent factor of greatest importance was the condition of the slab, its cleanness, roughness, and strength or soundness. "If the surface was clean, slightly rough, and free of a weak outer skin, good bond was generally obtained; otherwise relatively poor bond was obtained."

Inasmuch as heavy duty floor construction requires, for best results, a zero slump mix it follows that most concrete of this nature is job mixed in paddle or pan mixers, as Dr. Li suggests. The quantity of materials is relatively small due to the thinness of the wearing course. Uniformity of the mix is quite desirable and necessary. When bulk materials are used and batched by weight the utmost care must be exercised or segregation and moisture

*ACI JOURNAL, V. 30, No. 8, Feb. 1959 (*Proceedings* V. 55), p. 879. Disc. 55-57 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

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variation of the material will become apparent. If conveniently available it seems most desirable to use supplier dried, graded, sacked aggregate. Whereas this type of material usually is greater in first costs than bulk material, this will probably be more than offset because of the following reasons:

1. Much less waste of the aggregates, easier handling, and storage.
2. Labor saved at mixer by batching by sack rather than operating scales for weight batching.
3. Greater uniformity of mix by absolute control of moisture and uniformity of gradation resulting in easier finishing operations and more satisfactory surfaces.
4. Less likelihood of aggregate contamination.

Dr. Li advises that the joints in the wearing course should be formed carefully by sawing and should coincide with the joints in the base slab. One means we have seen used to successfully achieve the same result is to saw the joints in the base slab and insert metal strips so that the top of the strip would be the desired finished grade of the wearing surface. These strips are grouted into place and used as strike-off forms for the wearing course. The wearing course is concreted in "checker board" fashion, thus when one group of sections has sufficiently hardened the metal strips are removed and the next sections concreted against the edges of those placed previously. All edges are finished with a ⅛-in. edging tool. Floors done in this manner have given excellent service with no signs of deterioration at the joints. While this method may seem involved it should be remembered that if extremely hard aggregates have been used, sawing the joints in the wearing course will be difficult and costly.

We should like to add to Dr. Li's section on weather precautions with some mention of the potential effect of CO_2 from improperly vented heaters used in floor construction during cold weather. In the ACI JOURNAL Kauer and Freeman[15] said that their tests indicated that fresh concrete exposed to CO_2 resulting from the use of salamanders or other heaters not vented to the outside leave soft surfaces. These will be of various depths depending on the concentration of CO_2, the temperature at which the concrete is cured, and the humidities under which it was cured. It was also found that chemically produced hardners had no appreciable effect on the damaged surface; the only remedy for the soft film was grinding.

It has also been observed that even in heated enclosures in which the heating units have been properly vented, dusty, soft surfaces frequently occur. Investigation has disclosed that even though the mix was properly heated at the time of placing, these relatively thin slabs immediately lost their heat, thus resulting in the concrete remaining "dormant" for long periods. Final finishing has often been delayed as much as 10 to 12 hr. This has resulted in surface carbonation even though proper heater ventilation practices had been followed. It is therefore recommended that the concrete mix for floors which are to be placed under cold weather conditions be provided with an accelerator in order that final finishing not be delayed longer than 6 hr after placing. This may be accomplished by the use of 1 to

2 percent of calcium chloride or high-early-strength cement. The early application of a membrane curing media is also helpful in reducing the effects of the surface carbonation. Inasmuch as CO_2 is soluble in water wet curing media alone does not provide adequate protection.

Two-course construction is also applicable to bridge decks. Their useful life will be extended if they are equipped with a wear-resistant, durable concrete surface. This is particularly true in winter climates of frequent freeze-thaw cycles where the use of ice removal chemicals and tire chains has a tendency to spall bridge deck floors.

Dr. Li's article indicated that to obtain durable wear-resistant floors it is necessary to obtain a nonporous dense concrete on the surface.

This is particularly difficult to obtain in normal monolithic bridge deck placements. The water-cement ratio of the structural concrete for strength requirements is usually considerably above that which would be desirable for surface durability and abrasion requirements. Porosity of the surface concrete is further aggravated by the necessity of considerable vibration and manipulation of relative high slump concrete to get proper placement through the reinforcement, thus increasing the water and laitance gain to the surface. It seems highly feasible and desirable to construct bridge decks in two courses —the structural slab being struck off 1½ in. below finished grade, and a low water-cement ratio, low slump concrete wearing course being placed within 1 to 2 hr after the structural slab strike-off. If there is any question concerning the abrasion resistance and durability of local aggregates it would be wise to import aggregates of known desirable qualities for the concrete wearing course.

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By A. B. FOWLER*

This article covers two-course construction of floors or resurfacing floors, platforms, and aprons subject to heavy traffic or to severe use from the handling of heavy material.

From my own experience I am much opposed to two-course construction, except where absolutely necessary. Experience dictates monolithic construction is superior to two-course work. A metallic or other similar hardening agent can be applied and worked into the finished surface to increase its resistance to heavy traffic or severe abrasion.

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Two-course work is sometimes necessary for special jobs and resurfacing and therefore has a useful purpose. On new work, the top course should be placed while the subbase is still green, preferably within 24 hr after the subbase has been placed. On resurfacing projects, after the base slab has been prepared as recommended, special precautions should be taken to assure that all accumulated powdered dust has been removed; a 10 percent solution of muriatic acid, strong washing soda, or tri-sodium phosphate is helpful in this cleaning operation.

Under the subheading of "Cement," p. 882, it is noted that Type II cement has been eliminated from consideration for floor construction. This type of cement, it would seem to me, would be advantageous due to its moderate heat of hydration and slower setting qualities and is being used exclusively over all other types of cement whenever obtainable on all work under my jurisdiction. Type III cement, which is a high-early-strength cement, should be avoided except where it is necessary to put the floor in service at the earliest date possible and when used, due to its early setting qualities, special care must be given for proper and early curing.

Under "Water-Cement Ratio and Proportioning," p. 882, it is recommended that the total water in the mix be held between 3½ to 4 gal. per cu ft of cement. Such rigid limitations, due to the extreme difficulty of placing, prohibits its use because of productive and economical reasons.

"Air-Entraining or Wetting Agents," p. 883, recommends 3 to 6 percent entrained air. My experience on trowel finished floors indicates that it should not exceed 3 percent or be eliminated all together. This is due to the difficulty experienced in finishing, as the entrained air has a tendency to roll or ball while troweling, making it difficult to get a satisfactory finish.

"Corrections," p. 884, states that full bags of cement per batch should be used. This I am in accord with except that bulk cement is in common use today in which case cement would be weighed.

"Weather Precautions," p. 887, states that if during the progress of work the temperature is, or will within 24 hr, drop to 40 F, the water and aggregate should be heated. The anticipated weather drop 24 hr after concrete has been placed should not have any bearing on the temperature of the concrete at the time it is placed. It would seem that the emphasis should be on protecting the concrete at a reasonable temperature rather than heating of the aggregates.

It also seems questionable whether bituminous seals or mastic mortar surface treatment has much value for wear resistance.

By DANIEL A. GUNTIN*

Dr. Li is to be commended on an excellent treatise on those important factors and fundamentals of durable concrete wearing surfaces.

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Among the many significant comments in his review, we find two sections; one pertaining to the use of admixtures and another dealing with surface treatments of exceptional interest. Dr. Li, in our opinion, shows a penetrating knowledge of two sometimes misinterpreted topics. We refer to the specific advantages inherent in the use of certain admixtures and the values imparted to wearing surfaces by selective concrete floor treatments.

Apart from air entrainment, the properties of which are quite well known, the use of selective admixtures for purposes of water ratio reduction, set control, and workability, not to mention increased strength, especially flexural, reflects a growing trend in the design and production of modern day flooring. While the use of such admixtures may not be dictated for every installation, their advantages should be realized and used as job conditions warrant. For example, we have found that the use of a retarding densifier imparts valuable characteristics to a given mix beyond that of lower water requirements, higher strengths, and improved workability.

Oft times a floor slab or topping has a tendency to set at a faster rate in those areas adjacent to columns or walls. On certain installations this problem can be quite serious. Our experience has indicated that controlled retardation alleviates immeasurably the finishing problem. This common experience is not restricted as we know, to hot weather concreting schedules.

The values of a retarder are exemplified when aggregate compounds such as selected iron are to be installed by surface application techniques. Should the application or "shakes" of the aggregate be improperly timed to coincide with the required plastic state of the concrete, and this may occur for a number of reasons, severe difficulties can arise. Finishing may prove difficult, expensive, or may prove incapable of complying with original specifications. Controlled retardation under these conditions provides that needed assurance and safeguard for proper installation. Our comments here are limited to a retarder; however, different problems show that important advantages are also obtainable by the selective acceleration of set through the use of an appropriate admixture while still obtaining the benefits of water economy, workability, density, etc.

Dr. Li also referred to various groups of concrete floor treatments. This particular phase of our industry, although it enjoys a status of instant recognition as a necessity, does contain a number of facets not completely appreciated. We note, for example, that Dr. Li overlooked a group which many consider indispensible. These are the aggregate compounds, which are generally installed into the still plastic surface of the floor. Specifically, iron, quartz, aluminum oxide, and silicon carbide compounds, all of which have been widely used. They have had long use and service for heavy duty and nonslip purposes. The field results prove their value, standing up to all sorts of exposures with excellent performances.

We feel that the theory of relying on a few specific treatments for all exposures has lapsed. Indicative of this has been the development of diverse materials to meet modern floor needs. Although it is somewhat natural to

experience some overlapping of function, each type of treatment is specific in its original concept. That is, the treatment is meant to impart to a concrete surface an essential and desirable characteristic that is not *usually* associated with a property of the flooring. Thus, for example, the use of iron aggregate is desirable when electrical conductivity or spark-proofing is a requirement; aluminum oxide or silicon carbide when nonslip factors arise; and the silicofluorides where erosion resistance or surface density is needed.

As a result, the use of aggregate compounds or "hardeners" as they are commonly referred to, are very much a part of the technological considerations examined by Dr. Li. The significance of this to the designer or specifier is enormous. For he now has a valuable array of materials to supplement his carefully planned floor design. Thoughtful selection of one or perhaps a specific combination can enable him to obtain the desired end result at a minimum of cost and with assurance of maximum durability.

This particular phase of our industry is growing at a fast rate, stimulated as it is by the development of newer resin plastics, and techniques. Work definitely is needed on the subject of nomenclature and standards. In this respect, the proposed work of ACI Committee 402 on concrete floor treatments will be of considerable value as an adjunct to the recommendations and comments of Dr. Li.

AUTHOR'S CLOSURE

The author is grateful to Messrs. Elder, Fowler, and Guntin for the interesting discussions they have contributed to this apparently dull subject. Their valuable comments deserve analytical responses grouped categorically.

TWO-COURSE VERSUS MONOLITHIC CONSTRUCTION

Messrs. Elder and Guntin's hearty approval of two-course construction to produce economically durable concrete wearing surfaces that will endure severe conditions for a long period of service with little or no maintenance, does not, in any way, contradict with Mr. Fowler's view of "being much opposed to two-course construction except where absolutely necessary." It is just where absolutely necessary, that is, where wear-resistant concrete is indispensable, that the use of two-course construction is advocated.

Monolithic construction would be superior to two-course work, as Mr. Fowler contends, if wear-resistant quality were not the primary requisite. His further argument that a metallic or other similar hardening agent can be applied and worked into the finished surface to increase its resistance to heavy traffic or severe abrasion, appears true only during the early stage of service life. As age increases, experiences based on performance records invariably show that:

1. Metal grids result in unequal wear of the metal and concrete surfaces which in turn produce an uneven surface.

2. Hardening agents are not permanent, need periodic renewal, and do not serve as a panacea.
3. Unless a two-course construction is used, the renewal of a monolithic construction with metallic or other hardening agent makes it uneconomical and inconvenient.

Mr. Fowler also admits: "Two-course work is sometimes necessary for special jobs and resurfacing and therefore has a useful purpose." These special jobs are as enumerated in the beginning of the author's paper.

The decisive criterion for adopting a two-course construction or a monolithic construction depends, in the ultimate analysis, on whichever will require the least annual cost, or capitalized cost, when all the first cost, maintenance cost, and renewal cost are taken into consideration.

CONSTRUCTION SPECIFICATIONS

Mr. Elder rightly stresses that the success of such construction begins with the specifications concerning the base slab and the wearing surface. It was for providing a basic reference for preparing such specifications that the paper was written. The discussers have contributed much in elucidating many a salient point. Conventionally, specification writers are inclined to emphasize the best available for materials, methods, workmanship, and procedures. In a given project, however, it is not the best available but the best justifiable under a given set of funtional requirements and economical limitations that should dictate the stipulations of the specifications. Despite this *a priori* fact, "with respect to the wearing surface," as Mr. Elder judicially puts it, "the material costs are such a relatively small proportion of the over-all costs that the best available should be specified and used."

PREPARATION OF BASE SLAB AND PLACING OF WEARING COURSE

Under preparation of base slab and again under placing, the paper covers all possible conditions that may be encountered, with each individual treatment really in line with the comments of the discussers.

On new work

Mr. Fowler suggests that the top course should be placed while the subbase is still green, preferably within 24 hr after the subbase has been placed. The paper first provides procedures for preparing the base slab when the wearing course is to be placed the same day as the base slab; then gives additional procedures if the wearing course is placed as soon as the base slab has become sufficiently firm to withstand foot pressure, or has set sufficiently so that water does not rise to the surface; and finally states that a convenient procedure is to place the base slab in the afternoon and the wearing course the next morning. Thus, the author not only prefers the placing of wearing course on new work within 24 hr after placing the base slab, but also provides detailed procedures according to the degree of greenness of the base slab.

On partially hardened base slab

Mr. Elder refers the author's wetting of partially hardened base slab on new work to the test results of Felt concerning bond in resurfacing and patching concrete pavement. In this connection, the author posed two requirements in his paper: (1) thoroughly wetting the base slab just prior to placing the wearing course, and (2) brooming onto the surface of the base slab a thin coat of neat cement slurry or grout for a short distance ahead of the topping. The wetting is to eliminate absorption by the base slab of water from the zero slump mix of the wearing-course concrete, while the cement slurry will serve to insure the bond. "If the surface was clean, slightly rough, and free of a weak outer skin," according to Mr. Felt, "good bond was generally obtained" in his tests on resurfacing and patching concrete pavement. Presumably, Mr. Felt did not use zero slump mix of concrete in his tests in which case some absorption of water by the patched concrete from the patching concrete should prove to be beneficial.

In the case of zero slump wearing-course concrete, neither absorption of water from within by the base slab nor rapid evaporation of water from without is desirable; hence wetting of the base slab before placing the wearing course and wet curing after its placement. It is desired to have just sufficient wetting to eliminate absorption, and therefore no "minute pools" of water should be left standing on the wetted surface. On partially hardened base slab, this measure is necessary, because the partially hardened concrete is capable of reabsorbing water. Even a thoroughly cleaned dry surface may give adequate bond, when the strength of concrete is strictly controlled by water-cement ratio, it still proves to be good practice to wet the cleaned surface to keep the water content of the fresh concrete unaltered.

On well matured base slab

It is again principally for the purpose of eliminating absorption from the zero slump concrete wearing course that the paper states when the wearing course is to be laid over a well matured base slab, the latter should be soaked with water overnight or for an equivalent length of time. It is not aimed for increasing bond. Under this condition, bond with hardened base slab, as the paper states, is achieved by chipping, cleaning, and thorough scrubbing.

In fact, Felt's tests on resurfacing and patching concrete pavement is a case of resurfacing aged monolithic concrete pavement whose rugged and cleaned surface would provide sufficient bond if of sound concrete. Furthermore, when seasoned sound concrete is to be patched it has little capacity to absorb water from the fresh patching concrete. Felt achieved bond through roughness. In the case of two-course construction, the author attains it by chipping and scrubbing.

On resurfacing

In cleaning the subfloor for resurfacing work, Mr. Fowler suggests the use of a 10 percent solution of muriatic acid, strong washing soda, or tri-

sodium phosphate to assure the removal of all accumulated powdered dust. This is indeed effective, but the author prefers a thorough washing and scrubbing if the use of acids or alkalies can be avoided. Since both acids and alkalies will react with concrete causing the latter to disintegrate, their use must be followed by thorough washing.

The paper is primarily concerned with new construction, and hence resurfacing was only touched upon to complement the context. To keep this closure within reasonable length, the author deems it advisable to avoid detailed treatment of the entire subject of resurfacing.

INGREDIENTS

Cement

Type II cement is primarily used for construction that is exposed to a moderate sulfate action, or where moderate heat of hydration is required. Because of these special properties, which are not generally required, the paper states that Type II cement, "may usually" be eliminated from consideration, but not entirely as Mr. Fowler has commented. However, Mr. Fowler's experience with Type II cement is worthy of note. His requisite for proper curing when Type III cement is used, has been covered in the paper under curing.

Water-cement ratio

While, as Mr. Fowler remarks, to hold the water-cement ratio to not over 4 gal. per cu ft of cement causes difficulty in placing, this stipulation, nevertheless, constitutes the gist of the whole problem of achieving a strong and durable wearing course. Practical ways to attain this water-cement ratio have been given in the paper under "Water-Cement Ratio and Proportioning." To provide high wear resistance and durability and hence long range economy, this specification is indispensable. Most contractors from the viewpoint of production and economy would agree with Mr. Fowler. But contractors in the specific field of wear-resistant concrete construction can make this specification of 4 gal. per sack of cement an asset by producing a high-quality work. Mr. Elder has already adopted this water-cement ratio in advance in his recommendations for heavy-duty concrete floor finish when power floats are used.

Air-entraining agent

Mr. Fowler's experience on trowel-finished floors indicates entrained air should not exceed 3 percent versus the author's recommended limit of 3 to 6 percent. He states that "the entrained air has a tendency to roll or ball while troweling, making it difficult to get a satisfactory finish." This difficulty would have been reduced if he had used a water-cement ratio as low as stipulated by the author and, in introducing an air-entraining agent, he had further reduced the total quantity of water in the mix, including that in the aggregate and in the air-entraining or wetting agent, and concurrently he had re-

duced the fine aggregate slightly by an amount equal to the net change in volume of air plus water so that the cement factor of the concrete would be kept constant.

Aggregates

Clean, hard, tough, angular, well graded, and dried aggregates (fine and coarse) are absolutely essential to producing wear-resistant, durable concrete for the wearing course. Mr. Elder's suggestion of using supplier dried, graded, and sacked aggregates, is conducive to economy for the reasons he cited.

Quartz has a hardness of 7 in Mohs' scale. Granite has quartz as its essential mineral. They are the hardest as aggregates. Basalt and traprock, or diorite, though next in hardness, can be processed as excellent aggregates. Where they are not locally available, it is justified to ship them from a distant supply source. On the recent topping job of the Alabama State Dock at Mobile, Ala., the traprock aggregates were imported from Iron Mountain, Mo.

To show the typical properties of hard aggregates, the following excerpt is extracted from a Pittsburgh Testing Laboratory Report on Iron Mountain traprock:

Chemical constituents, percent:			
Silicon oxide	57.94	Abrasion, percent of wear (Deval)	2.4
Aluminum oxide	7.83		
Iron oxide	21.53	Abrasion, Los Angeles (ASTM)	13.2
Calcium oxide	5.72		
Magnesium oxide	6.32	Hardness (Department of Ag. No. 347)	19.6
Sodium-potassium oxides	0.51		
Physical properties:		Fusion Point (ASTM C 24-35), deg F	2000
Specific gravity	2.94—3.00		
Absorption, in lb per cu ft	0.459	Solubility, 10 percent H_2SO_4 at 100 F, percent	1.43
Absorption, percentage	0.25		
Toughness (ASTM)		Solubility, 10 percent HCl at 100 F, percent	0.71
With grain	19		
Against grain	30		

Spalling: No tendency to spall after 10 cycles of heating to 2000 F on sudden cooling to 500 F.

Not only are the aggregates resistant to wear, abrasion, but also resistant to corrosion, intense heat, and temperature changes.

Selective admixtures

Mr. Guntin's forward outlook on the use and advantages of selective admixtures should have a far-reaching influence on the progress of modern concrete technology. The use of such admixtures for purposes of water-cement ratio reduction, improved workability, set control, increase in density, and increased strength, is growing and has had rewarding results. Several types of retarders and accelerators are available. Mr. Guntin confirms the merits of a mettallo-organic type retarding densifier. There is a crying need for keeping and publishing impartial performance records on job applications of such

admixtures. In them lies an additional potential benefit to the conventional ingredients in the production of durable concrete. For best results, optimum amount, appropriate moment, and proper procedure are important in handling the sensitive admixtures. Mr. Guntin's account of his experience with retarders constitutes a valuable contribution to the subject.

WEIGHING AND CORRECTIONS

Under "Weighing," the paper proposes to weigh all constituents which, of course, include bulk cement if used. This agrees with Mr. Fowler's suggestion that bulk cement should be weighed. To extend this precaution to all ingredients, "when bulk materials are used and batched by weight," states Mr. Elder, "the utmost care must be exercised, or segregation and moisture variation of the material will become apparent." Careful weighing alone is not sufficient. The author states how corrections should be made. Mr. Fowler agrees with the author in using full bags of cement per batch. Should it be necessary to use less than a bag of cement per batch, the author further calls for correction for the increase in volume of cement due to aeration by disturbance.

WEATHER PRECAUTIONS

Mr. Fowler's statement: "The anticipated weather drop 24 hr after concrete has been placed should not have any bearing on the temperature of the concrete at the time it is placed," really confirms instead of corrects the author's first paragraph under this heading. To quote from the paper, "If during the progress of work the temperature *is*, or will *within* 24 hr drop to 40 F, the water and aggregate should be heated and *precautions* taken to keep the temperature of the *concrete* above 70 F for at least 3 days or above 50 F for at least 5 days." The precautions are for *within* rather than after 24 hr, and they are directed to protecting the concrete. Compare then with Mr. Fowler's further statement: "It would seem that the emphasis should be put on protecting the concrete at a reasonable temperature rather than heating of the aggregates." Heating of the aggregates is also indispensable "if during the progress of work the temperature *is* in the neighborhood of 40 F."

Mr. Elder has added an excellent passage on weather precautions by mentioning the potential effect of CO_2 from improperly vented heaters used in floor construction during cold weather, and by referring to surface carbonation even though proper heater ventilation practice had been followed. The author maintains the practice of avoiding the use of salamanders or other heaters as far as possible. He kept silent in this regard in writing the paper, as there is no ideal recommendation that could be made even under properly and carefully ventilated conditions. He agrees with Mr. Elder in providing the concrete mix for floors to be placed under cold weather conditions with an accelerator, and in the early application of a membrane curing media, for reasons propounded by Mr. Elder.

JOINTING

Mr. Elder provides a commendable means to achieve good wearing-course joints by sawing the joints in the base slab, by inserting to grade and grouting into place interim metal strips of checker board fashion as strike-off forms for the wearing course, and by removing them when one group of sections has sufficiently hardened.

Though minor in workmanship detail as it may seem, it is highly desirable (1) to prevent moisture loss from joints during curing, and (2) to keep them clean of foreign matter until sealed. The former is essential to good curing of sawed or formed joints in green concrete. The latter eliminates resawing, blowing of joint with compressed air, wire brushing, or hooking out of incompressible materials lodged in joints. These two objectives may be accomplished by the application of concrete joint curing tape.

It consists of two ribbons of special adhesive, applied to a polyethylene tape, equidistant from center, and protected by a glassine backing. During application this backing is snapped off and the tape pressure-applied so as to straddle the joint. It can be applied within 30 min after sawing and water-flushing the joint. The moisture which collects on the underside of the tape within 10 min or less is the original moisture in the concrete constituting the very essence to the proper curing of the joint, and will be retained by the tape until the joint sealant has been applied. Proprietary seal tapes of this category are available in the market.

BRIDGE DECKS

When the paper was written, the author restricted its scope to heavy-duty floors, factory working areas, platforms at transportation depots, and aprons and floors at waterfront facilities. Two-course construction is naturally also applicable, as Mr. Elder has pointed out, to bridge decks. This is particularly true in the heaviest traffic areas located where the damaging effects of severe winter weather and of climatic extremes are encountered.

Though, in northern states, a wearing course has often been used on bridge decks, it has rarely been of wear-resistant quality. It is highly desirable, as is suggested by Mr. Elder, to construct bridge decks in two courses, the permanent strength slab, and a wearing surface of abrasion-resistant aggregates, low water-cement ratio, and low-slump concrete.

The durability of bridge decks may also be increased by the use of silicones. The most effective and most economical type of silicone for this purpose is the water soluble product, sodium methyl siliconate, at an optimum concentration of 2 percent silicone solids.

Two factors contribute to silicone protection: (1) the silicones' inherent repellency of water; (2) the ability of the silicone chemical, in diluted solution, to penetrate into the pores of masonry and to line those pores with silicone.

Requirements for water soluble silicones have recently been included in the New York State Public Works Specifications, and in the specifications of the State of Wisconsin Highway Department. Previously, in addition to laboratory tests, field test applications were made in several areas through the cooperation of state highway and toll road authorities. Though they are mainly used for improving the durability of concrete highway pavement, a 2 percent silicone solids treatment was applied to all of the curbing, except one small section, of a New York highway bridge. Two years later, all of this treated non-air-entrained concrete curbing was in perfect condition, but the untreated section was seriously deteriorated by spalling.

Those who are interested in silicone treatment of concrete pavements are referred to an excellent article entitled "Extending Concrete Highway Durability and Light Reflectance with Silicones," by Harold L. Cahn and Royal V. Mackey, Jr., ASTM Bulletin No. 235, Jan. 1959, pp. 37-42.

TUNNEL ROADWAY DECKS

To complement Mr. Elder's extension of two-course construction to bridge decks, the author wishes to further recommend it for tunnel roadway decks. A high wear-resistant surface course is extremely desirable in the roadway decks of vehicular tunnels. Here, both from the traffic standpoint and from the renewal standpoint, a wear-resistant two-course construction will minimize maintenance, prolong necessary renewal after much longer period of service, and facilitate the indispensable renewal with greater ease and convenience. And for still greater ease of renewal, precast wear-resistant concrete flooring units may be introduced to facilitate paving and to put into service at minimum time.

SURFACE TREATMENTS

Aggregate compounds

Mr. Guntin has elaborated on "aggregate compounds" for concrete floor treatments. They have been, indeed, widely used, and are indispensable under certain conditions. It was because some of them have been applied for other purposes, such as wrought iron aggregate for electrical conductivity or spark-proofing, aluminum oxide or silicon carbide for nonslip, that the author had purposely not included them in writing the paper. Its scope was confined to wear-resistant construction, as the title indicates, rather than the entire province of floor construction and surface treatments.

In examining the physical and chemical characteristics of aggregate compounds, the author found that the reaction between the calcium hydroxide in portland cement and the metallic silico-fluorides tends to densify and harden the concrete by precipitating compounds in the pores—a process that is conducive to wear-resistance. Hence, metallic silico-fluorides, consisting of magnesium silico-fluoride, zinc silico-fluoride, lead or aluminum silico-fluoride, as a composite surface treatment material, together with the proper method of application are given in the author's paper.

The central issue of the paper lies in how to achieve wear-resistant and durable concrete. The use of aggregate compounds or "hardeners" is a part of the issue, as Mr. Guntin so comments, but it is limited to increasing abrasion resistance, postponing the exposure of internal pores, acting as a moisture barrier, and preventing dust formation. The author states: "There are many proprietary hardeners and dustproofers available." It was left to the user to find out the comparative merits and economics of them from actual performance service records under conditions similar to those a user has in his project. "Each will serve its purpose to a certain extent under normal conditions," continues the author, "each will increase the serviceability of a good concrete, but none is a substitute for good concrete." He further adds, "They are, however, not permanent and need periodic renewal. —There is no surface treatment which can be depended on as a panacea."

Mr. Guntin emphasizes that the significance of the use of aggregate compounds or hardeners to the designer or specifier is enormous. He culminates with the statement: "Thoughtful selection of one or perhaps a specific combination can enable him (designer or specifier) to obtain the desired end result at a minimum of cost and with assurance of maximum durability." This is a promise to be welcomed. But engineers would like to have conclusive service performance records together with precise conditions under which the records were observed, on all the aggregate compounds or hardeners so far patented.

Resin plastics

Mr. Guntin further refers to resin plactics. The author covers briefly this group which includes those containing polyvinyl chloride and acetate copolymer, phenol formaldehyde, cashew nut resin, furane, polystyrene, and epoxide, alkyd, or coumarone-type resins.

Bituminous seal and mastic mortar

Mr. Fowler questions whether bituminous seal or mastic mortar surface treatment has much value for wear resistance.

One of the essentials for a durable wearing course is to have "dense concrete." "Maximum density implies minimum porosity and permeability." The use of bituminous seal or mastic mortar tends to seal the pores, acts as a moisture barrier, and prevents dust formation out of the concrete wearing course. Thus the elimination of the exposure of internal pores together with a moisture barrier will protect the portland cement concrete from suffering deterioration of the constituents of the hydrated cement.

Aggregates for wear-resistant concrete require hard sand for the fine, and hard crushed stone such as basalt and traprock, granite, quartz, or diorite for the coarse. The essential minerals for these aggregates consist of quartz (7), orthoclase (6), plagioclase (6), amphiboles (5 to 6), and pyroxenes (5 to 6), their respective hardness in Mohs' scale being indicated within the parentheses. Any minute breaking of particles will cause dust formation. Siliceous dust is on the hardness scale of as high as 7. Such dust will act as a grinding agent

under truck wheels, accelerating mechanical wear. Bituminous seal or mastic mortar will intercept such grinding action on the wearing course concrete.

Nomenclature and standards

Mr. Guntin's final suggestion on the need for nomenclature and standards in surface treatments of concrete floors, deserve the Institute's earliest due consideration. It is only natural to include this suggestion in the proposed work of ACI Committee 402.

POSTLUDE

In closing, the author wishes to thank all of the discussers for their enthusiasm in contributing so much valuable comment. It has been his greatest pleasure to carefully read the discussions and to ponder further on the subject while writing this closure. If the paper has supplied a need at this time, has served a framework for further development, has provoked or renewed scientific interest in the important question of wear-resistant concrete surfacing on which every one may put his feet now and then, or has clarified hitherto hazy points, the author will feel more than duly rewarded.

Discussion of a paper by B. G. Singh:

Specific Surface of Aggregates Applied to Mix Proportioning*

By K. W. DAY, ROMAN MALINOWSKI, and AUTHOR

By K. W. DAY†

I have been using for the past 5 years a method of mix proportioning similar to that proposed by Dr. Singh. Although I have not had the opportunity to perform laboratory experiments to any great extent, the system has successfully produced answers to many difficult mix proportioning problems in commercial applications and my experiences may be of interest.

I have considered two separate factors in attempting each design:

(1) The water requirement (and therefore the cement requirement) and also the segregation resistance of a mix is almost entirely dependent on the over-all mix specific surface. For any given use of the concrete a suitable minimum specific surface can be selected. This specific surface will be low (16 to 19 sq cm per g) for very dry concretes with placing conditions giving no appreciable risk of segregation and high (22 to 27 sq cm per g) for wetter mixes placed, for example, in thin vertical molds. Mixes of higher specific surface than the minimum required will be suitable for placing but will not achieve the maximum cement economy and may not be suitable for high strength work.

(2) The amount of matrix available must be sufficient to give a dense concrete. This may require more sand than is strictly desirable from the specific surface point of view and either the loss in cement economy must be accepted or alternative aggregates must be used, whichever gives the greater over-all economy.

In this latter respect there is a serious omission in Dr. Singh's list of factors affecting the coarse to fine aggregate ratio. The factor omitted is the density of packing of the coarse aggregate. It is clear that the lesser the volume remaining to be taken up by matrix the more likely it becomes that maximum cement economy can be achieved by using the minimum specific surface suitable from a segregation point of view.

Dr. Singh's relationship between aggregate-cement ratio and desirable specific surface, although based on experimental evidence, may be misleading since it is a result of extending the particular into the general. The real requirements for a satisfactory lean mix are high density of packing of coarse aggregate and a sand having a sufficient proportion of fines [passing a 52 or even 100 mesh British standard sieve (BSS)] to enable the cement paste to

*ACI JOURNAL, V. 30, No. 8, Feb. 1959 (*Proceedings* V. 55), p. 893. Disc. 55-58 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

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fill the remaining voids. Although this creates a situation in which it is often difficult to achieve the desirable minimum specific surface, it does not make it any less desirable.

Another reason for the upward slope of the graphs given in Dr. Singh's paper is that the effect of the cement itself on the segregation tendency (or effective specific surface) is large. It is not however large enough to explain entirely the slope shown.

A point which puzzles many engineers on meeting specific surface theory for the first time is that cement, with a specific surface ten times that of the finest sand, does not entirely swamp the effect of aggregate specific surface. Without delving into the reason for this it is quite clear that the "effective specific surface" of cement in this context is much less than its actual specific surface. It is not so widely realized that this effect also extends to the finer sand fractions. What is actually being sought is a "water factor" for the various sieve fractions and specific surface is of interest only because it more closely represents this water factor than does any other known property (such as fineness modulus). The water factor of a sieve fraction may be defined as the amount of water which would have to be added, together with one gram of that sieve fraction, to a concrete mix of some standard workability, in order that the addition may leave unchanged that workability. The finer sieve fractions of the mix (including the cement) have a smaller effect on water factor or effective specific surface than is indicated by their true specific surface.

In applying specific surface theory to practical mix proportioning I have made use of the actual measured specific surface of the various sieve fractions obtained by Newman and Teychenné in the course of their preparations for a paper[6] (values not published in that paper). These values have been modified[7] as a result of practical experience to more truly represent water factors. These values have proved satisfactory on aggregates and sands from many different sources, leading to the conclusion that the differences in specific surface due to different particle shape are not fully reflected in the water factors of the aggregates.

To illustrate the wide range of applicability of the method, two examples are given:

(1) On a bridge precasting site at Canberra (Australian Capital Territory) the only local sand was extremely coarse. A specific surface of 18 sq cm per g was considered appropriate for the work (a rich, high strength mix; incidentally this value accords well with Dr. Singh's graph) and this gave the required sand fraction as 47 percent of the total aggregates. Although this seemed impossibly high, no objection could be found in the second requirement that sufficient sand should be available to fill the voids in the coarse aggregate; sufficient cement was required on strength grounds to fill the voids in the coarse sand and sand was cheaper than coarse aggregate. A trial mix was therefore prepared and was highly successful.

(2) On a second bridge site at Hobart (Tasmania) the only local sands had virtually nothing retained on a No. 25 BSS sieve and there was a suggestion to ship by rail a coarser sand from over 100 miles away as it was felt that high strength concrete could

not be made with such fine sand. An examination of specific surfaces (by sieve analysis) revealed that only 15 percent of sand was desirable. It was discovered that by combining 10 percent of ⅜ to ¼-in. crushed stone with 75 percent of a single-sized ¾-in. crushed stone a dense coarse aggregate with fairly low specific surface resulted. When this was combined with the sand and cement a usable concrete resulted although it did require careful handling to prevent segregation. An increased sand proportion was tried to reduce the segregation tendency but did not yield the necessary strength (extra cement did not give additional strength below a 3½ to 1 aggregate-cement ratio). The mix as first proportioned was in fact used for over 200 prestressed 40-ft span beams.

In both the above jobs a minimum cylinder strength of 5000 psi at 28 days was called for and obtained with a 1-in. slump concrete.

At the other end of the aggregate-cement ratio scale, the same theory, and the same mix specific surface (18 sq cm per g) has been used on earth-dry mixes of 7 to 1 aggregate-cement ratio compacted under almost ideal conditions with heavy external vibration to produce extremely strong concrete (8000 psi cylinder strength at 28 days). This is all the more at variance with Dr. Singh's graph because aluminous cement (which is much more coarsely ground and therefore has much less effect on the total effective mix specific surface) was used.

In conclusion I do not wish to detract from Dr. Singh's excellent paper and hope that he will continue his work to produce a really comprehensive theory of mix proportioning. One most intriguing possibility is the discovery of a means of directly ascertaining the specific surface of sand sample by a method quicker than a sieve analysis. I have attempted this by means of a water permeability test but so far without success.

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By ROMAN MALINOWSKI*

The paper presented by Dr. Singh is a continuation of his previous two papers[3,4] and in it an attempt is made to apply the factor of specific surface of aggregates to mix proportioning. The limited space of the paper did not allow the author to critically review papers by Edwards[1] and Young[2] mentioned by him or the interesting method of Kennedy.[8]

REMARKS ABOUT MIX PROPORTIONING

Aspect of the method

The author limits the basis of calculating the mix proportions by using only accepted formulas. He gives no details about the kind of experiments

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performed, the data obtained, or corrections needed. The paper shows only the theoretical value of this method. Taking into consideration the diverse character of the ingredients of concrete and the difficulty of testing such properties of the aggregate as shape, roughness, density, and absorption (during mixing and during placing), the calculation of the mix based on theoretical formulas can only be a first *approximation* which needs experimental verification.

Basis of the method

The proportioning of a mix is based on: (a) water-cement ratio for a required strength; (b) index of consistency, described as compacting factor (CF); (c) formula for the quantity of cement; and (d) the formulas of the aggregate proportions.

As to the W/C, the author does not suggest a formula, although in Reference 4 he developed one of his own which was a function of specific surface.

Eq. (3) is an inherent part of the author's method and it relates the consistency of the concrete to weight factors such as: W/C, aggregate-cement ratio (N), standard consistency of the cement (P), and the specific surface factor (S_o in sq cm per g).

Eq. (4) is a transformation of Eq. (3) for $P = 0.28$. As it turns out the author uses, instead of his equations, graphs (Fig. 2 and 3) for finding N and S_o. The formulas and the graphs are applicable only for concrete made with gravel having a maximum size $D = \frac{3}{4}$ in. The formulas are thus limited and do not have universal usage.

The term of Zone 1 and 2 for continuous and gap graded aggregates, respectively, introduced in Fig. 2 has no equivalent in Eq. (3), so it limits the general character of the formula and the factor of specific surface. The deviations within the zones are considerable; for example, for $S_o = 30$ sq cm per g (in Zone 2) we receive $5.3 < N < 7.7$, which in identical values of W/C and CF gives a big deviation in the quantity of cement $\pm$ 50 kg per cu m.

In Eq. (5), which serves to calculate the quantity of cement in the concrete, the factor of specific surface S_o has not been taken into consideration.

The proportioning of aggregates is based on the factor S_o (for all ingredients) arrived from at Fig. 2 as well as on the factor of specific surface of aggregates derived from sieve analysis [Eq. (1), (2), (6), and (7)]. The surfaces obtained this way bear no relation to the actual properties of the ingredients and may thus change the actual value of the factor S_o.

The superiority of aggregates proportioned in this way, as compared to other methods which are also based on sieve analysis (fineness modulus, water-aggregate ratio after Bolomey or Stern[9]) is rather doubtful.

Both the method of mix proportioning and the formulas indicate their limited and approximate character. It is a pity that this method, having an experimental foundation, does not go on trying to confirm by trial its essential assumptions. Eq. (3) has considerable value and takes into account all

the factors necessary for the determination of the real properties of the aggregates.

REMARKS ON THE FACTOR OF SPECIFIC SURFACE S_o

Neither the fineness modulus nor the density factor define correctly the properties of aggregates. Many authors consider that the factor of specific surface, used as a physical value, may be more applicable for predicting the properties of fresh concrete (cohesion, adhesion, and consistency) and of hardened concrete (compressive, tensile, and flexural strength), and so it may be a better way of measuring the properties of aggregates.

As shown above, because of the diversity and shape of aggregate surfaces a theoretical determination of the quality of the aggregates cannot be done on the basis of only a sieve analysis. Therefore, the calculation of specific surface, based on sieve analysis, can be regarded only as a kind of fineness modulus.

Similarly the specific surface factor of aggregates S_o based on empirical formulas is only of an approximate value. Its practical importance is limited as it gives only an auxiliary proportioning factor.

Kuczynski[10] shows that the use of the fineness modulus together with the density factor gives a more reliable evaluation of the properties of aggregates. Kennedy[8] combines the theoretical specific surface factor (calculated from sieve analysis) with the density factor. Others[11] introduce the conception of the surface modulus as a quotient of the density and specific surface factor $M = D/S$.

All these factors have only an approximate value in view of a certain inaccuracy of sieve analysis and of determining the density.

EXPERIMENTAL DETERMINATION OF THE QUALITY OF AGGREGATES IN CONSISTENCY TEST

Only an experimental determination of the quality of the aggregates in the concrete can be regarded as accurate. This is fulfilled by measuring the consistency of the concrete.

The consistency of the concrete depends on the quantity and the consistency of the cement paste as well as on the properties of the aggregates. It is obvious that *a given quantity and consistency of cement paste and the consistency of concrete can determine the properties of the aggregates.* Thus for a determined quantity of ingredients the index of consistency may be a measure of the properties of the aggregates. The method of measuring consistency should be adapted to the kind of the concrete tested.

The principle of experimental determination of optional proportions of aggregates as a function of the consistency can be found in the works of Kluz and Eyman[12] and Skramtayew.[13] The last one recommends trial mixes for three proportions of aggregates containing the *same quantity of ingredients.* The best of these three mixes will be the one having the highest

density and the greatest workability. In this mix, surplus water may be eliminated when having the same consistency. *This concrete will usually prove the most dense.*

INDEX OF CONCRETE CONSISTENCY AS AN EXPERIMENTAL MEASURE OF THE QUALITY OF AGGREGATES

Starting with a given mix proportion of 300 kg per cu m of cement and 180 kg per cu m of water and using a water-cement ratio by weight for a standard consistency of $P = 0.28$, the properties of the aggregates are then determined. For an aggregate with a fixed $D_{max.}$ and a fixed density, the consistency test becomes a case of measuring its other properties.

These properties can be determined by means of graphs[14] and/or by using theoretical formulas, provided they take into account all the basic factors that influence the consistency.

The numerical determination of these properties for aggregates can be computed by using any formula for consistency, for example the formula by Stern[9] or the equation by Dr. Singh. It should be noted that Eq. (3) by Dr. Singh contains all the above mentioned factors, and this is true of only a few other methods.

The transformation of this equation gives the specific surface

$$S_o = \frac{\frac{W}{C} \cdot \frac{1}{CF} - 1.21\,P - 0.0604 \cdot N \cdot \frac{W}{C}}{0.0005} \text{ sq cm per g} \quad \ldots\ldots\ldots (8)$$

After rounding off the numerical cooefficients, we get

$$S_o = \left[\frac{W}{C}\left(\frac{1}{CF} - 0.06\,N\right) - 1.2\,P\right]\frac{2000}{N} \text{ sq cm per g} \quad \ldots\ldots\ldots (9)$$

Introducing standard conditions, such as $W/C = 0.6$, $N = 6$, and $P = 0.28$, we come to the formula

$$S_o = 200\left(\frac{1}{CF} - 0.92\right) \text{ sq cm per g} \quad \ldots\ldots\ldots\ldots\ldots (10)$$

This formula is also applicable for other standard conditions such as: $W/C = 0.7$, $N = 7$, $P = 0.28$; and $W/C = 0.8$, $N = 8$, $P = 0.28$.

The value of the compacting factor can be determined by test. For $0.75 < CF < 1.0$ we arrive at $80 > S_o > 14$ sq cm per g.

In this case, it is my opinion, there are no limits for the size of the aggregate or its character (continuous or gap graded). With the above results and data for aggregate size, specific weight, density, and absorption, a full picture is obtained. A factor arrived at in such a way should not be taken as

scientifically exact or of finite character. But by employing its use we can determine in a remarkable way the properties of aggregates.

It has not been the purpose of this discussion to solve the question of the specific surface of aggregates, which is certainly a difficult one. The purpose has been rather to stress the importance of the experiment in determining the quality of the aggregate.

These remarks may be useful when choosing an optimal aggregate mix proportion. They may also be of some meaning when a standard method of determining the properties of aggregates is considered.

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AUTHOR'S CLOSURE

I am pleased to know that Mr. Day has successfully used a similar method of proportioning for the past 5 years; the measure of agreement between his experience and the method as outlined is encouraging. My work described in this paper was done between 1950-52 and formed part of the report by Newman and Teychenné[6] and therefore should be in agreement. Mr. Day's field experience is of particular interest as I have not had the opportunity of applying this method to practical problems. However, there are a few points raised in the discussion that call for comment.

It is stated that: "The relationship between desirable specific surface and aggregate-cement ratio may be misleading for it extends from the particular into the general." This is not the impression I wished to convey, for it is clearly stated that the relationship "may have to be modified in the light of experience" and further "the method of proportioning put forward in this paper is restricted to the type and maximum aggregate size used." The material passing the 100 and 200 mesh sieves will have a higher specific surface and as such will cause an increase in the water-cement ratio to maintain a given consistency and if aggregate of the same specific surface gives the same consistency and strength for a given water-cement ratio, provided harshness is avoided (i.e., sufficient fines), then it is somewhat difficult to visualize the advantage of using material passing the 100 mesh sieve, at least for normal range of mixes. It has been shown that the 52-100 size material tends to reduce naturally entrained air* to a minimum and it is

*Singh, B. G., "Aggregate Grading Affects Air Entrainment," ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), pp. 803-810.

possible therefore to obtain high density without the use of material passing the 100 and 200 mesh sieves.

Mr. Day remarks that I have made "a serious omission" by not mentioning the density of packing of the coarse aggregate as a factor affecting the fine to coarse aggregate ratio. It is not easy to deduce how this conclusion was drawn since I listed factors that influence fine aggregate content for a given maximum size and shape on p. 895. I believe the particle size distribution of the coarse aggregate, even with relatively great variation, is of little practical significance in relation to requirement of fines.

The generalization that "the segregation resistance of a mix is almost entirely dependent on the over-all specific surface of the mix" is not easy to follow, as particle size distribution and consistency are both of significant importance in relation to segregation. Whereas it is true that mixes with more cement (higher over-all specific surface) may resist segregation more effectively, it is also true that two mixes of the same over-all specific surface may show different degrees of resistance to segregation, particularly if one of the aggregate is poorly graded and has two or more gaps. The successful use of aluminous cement with a lower specific surface, with an aggregate of lower specific surface than that suggested should not be held "to be more a variance." On the contrary the opposite could be true; because of its greater coarseness and range of particle size distribution it is more likely to avoid a gap in grading between the cement and the fines of the aggregate.

It is not clear from Mr. Day's discussion whether the 18 sq cm per g aggregate which he used successfully for a 7:1 mix was of a similar maximum size and shape. It is also not clear whether he used the constants supplied by Newman and Teychenné or what he calls his "water factor." If it is assumed that his maximum aggregate size and shape and his method of deriving the specific surface were similar then it would seem that for heavy vibration and earth-dry mixes a specific surface lower than that suggested, could be used with advantage.

As Eq. (4) relates consistency, as determined by the compacting factor, aggregate-cement ratio, and aggregate specific surface to water-cement ratio, it should be easy to assume a suitable specific surface for range of aggregate-cement ratios and then relate water-cement ratio to aggregate-cement ratio and consistency. However, it should be repeated that the equation is applicable for aggregate of similar size and shape to that described in the report. It should also be pointed out that the CF test becomes less and less applicable as the mix gets harsher and harsher. Thus with specific surface such as 18 sq cm per g in a 7:1 mix high compacting factor values will not be obtained experimentally. Under such conditions the estimated CF value may be taken as a wetness factor.

I am indebted to Professor Malinowski for making such a close study of the paper.

Efforts were made throughout to stress the limited quantitative application of the method. Admittedly, if such factors as shape, roughness, and

absorption were included in more comprehensive experiments it would be possible to include them in Eq. (3) to give it wider application. However, Professor Malinowski should not find it difficult to concede that the method is applicable for aggregates classified as "irregular" by BS-882:1947 for the factors mentioned will be significantly constant and trial mixes will become more and more important only as aggregates correspond less and less to this definition.

Professor Malinowski suggests that "the proportioning of aggregates in this way, compared to other methods is rather doubtful." It is easy to agree with him if he means that the method has doubtful application for other aggregate types. This was clearly not intended. The main advantages claimed for the method is that it is applicable to both continuous and gap graded aggregate and it is based on a physical property determined by tests and so capable of physical interpretation. As it can be shown that neither the fineness modulus (and any modification of it) nor Bolomey's method are applicable to gap graded aggregates and these are less descriptive of the aggregate properties than the specific surface, any rational method of design such as the one described, should have wider application. Mr. Day's experience may help to reduce further doubts.

To use Eq. (1) and (2), the specific surface of each size group must be determined before hand by the water permeability tests[5] whereas Eq. (6) and (7) (based on Young's surface modulus) are suggested for estimating specific surface values for other types of aggregates in the absence of proper tests. It is difficult to appreciate the statement that "specific surface based on the sieve analysis is a kind of fineness modulus," and it would appear that Professor Malinowski overlooked (a) that there is no reason why the sieve analysis should be less accurate when calculating (or estimating) specific surface compared to other indices of grading, and (b) the surface modulus is more closely related to specific surface than the fineness modulus.

The fact that no equivalent for Zones 1 and 2 (Fig. 2) exist in Eq. (3) or the absence of S_o from Eq. (5) should not be regarded as valid criticisms. Contrary to what Professor Malinowski tries to infer, Eq. (3) cannot be taken to give any indication of acceptable S_o values with respect to N. Eq. (3) obviously has limited application for it does not take into account all variables as he rightly points out.

Acceptable S_o values with respect to N is conditioned by other factors and independent of any variables in Eq. (3). As the specific surface of the sand increases, its volume required to give a desired S_o value decreases and could do so to a point when the voids in the concrete cannot be filled. Therefore as the sand becomes finer a higher S_o value is allowed to overcome harshness which is the explanation for Zones 1 and 2. A variation within a zone is permitted to allow for a variation in "fatness" to suit the conditions of the job. S_o should not be expected to appear in Eq. (5) when its suggested use is simply to calculate the quantity of each component that when combined will give the required consistency and strength.

Professor Malinowski goes on to state "for S_o = 30 sq cm per g (in Zone 2) we receive $5.3 < N < 7.7$, which for identical values of W/C and CF gives a big variation in the quantity of cement ± 50 kg per cu m." It is obvious that W/C and CF cannot be identical if N is varied. For example, for $W/C = 0.5$ and $S_c = 30$ sq cm per g, CF will be 0.86 for $N = 5.3$ and CF = 0.73 for $N = 7.7$. Thus the ± 50 kg per cu m variation found by Professor Malinowski is not a valid criticism. It simply shows that for a given W/C ratio a lean mix at a stiffer consistency will result in a saving in cement.

Professor Malinowski's statement that "a given quantity and consistency of cement paste and consistency of concrete can determine the properties of the aggregate" might not be taken seriously but he goes on to express S_o in terms of CF. For $N = 6$, $W/C = 0.6$ and $P = 0.28$ he obtains $S_o = 200\ [(1/CF) - 0.92]$ sq cm per g. He states that "this formula is applicable for other standard conditions $W/C = 0.7$, $N = 7$, $P = 0.28$, and $W/C = 0.8$, $N = 8$, $P = 0.28$ for various types of concrete." A simple calculation will show the 0.92 term varies from 0.90 for $N = 8$ to 0.97 for $N = 5$ for even his standard conditions and therefore his statement about the general application of the formula is in doubt.

The equation is used to calculate S_o for $0.75 < CF < 1.0$ and he obtains $80 > S_o > 14$ sq cm per g and he concludes that "In this case, it is my opinion, there are no limits for the size of aggregates or its character (continuous or gap graded)." Apart from the fact that this equation is in doubt even for Professor Malinowski's standard conditions, his interpretation is clearly erroneous for the only interpretation that can be given to the results obtained is that provided $N = 6$ and $W/C = 0.6$, CF will increase from 0.75 to 1.0 if S_o is decreased from 80 sq cm per g to 14 sq cm per g. As Eq. (3) was obtained for ¾-in. aggregate, and aggregate size is of importance, no amount of juggling could prove that it is applicable to other aggregate sizes.

Discussion of a paper by Warren A. Shaw and J. R. Allgood:

Blast Resistance of Reinforced Concrete Beams Influenced by Grade of Steel*

By M. J. GREAVES, K. E. McKEE, and AUTHORS

By M. J. GREAVES†

Although the authors have made a valuable contribution to the literature on blast resistance of reinforced concrete beams, much work remains to be done to collect the necessary physical data to define and extend the scope of application.

The authors state: "In most blast resistant construction the design objective is a structure whose deflection will be less than some specified limit under the design loading." This design objective is reasonable and sufficiently broad to cover most circumstances including situations where the deflections are elastic and at the other extreme, deflections which approach total failure. The authors' second criterion appears to encompass this design objective.

However, a loose application of the authors' first criterion by an inexperienced designer may lead to the questionable conclusion that a beam which fails has performed better than a beam which suffers no damage.

By K. E. McKEE‡

The authors are to be commended on their paper—the most rational approach published to date on the influence of the grade of reinforcing steel on the blast resistance of reinforced concrete beams. It was demonstrated that the ability of a beam to resist loads depends not only on the grade of steel but also on the ratio of load duration to the period of vibration and on the failure criteria. This shows a general fact relating to blast resistance—the "best" material or design may vary with several parameters and the selection should be based on a dynamic analysis for the specific problem of interest rather than on generalized finding.

The approach used by the authors makes use of data of modified static resistance. This is the general approach which commonly has been used in blast effects work. Ideally one would prefer to use the resistance-displacement which the beams exhibit when subjected to dynamic loads. Since

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some dynamic data is available,[2] consideration could be given to this data to determine if the resistance-displacement under static loads is a suitable representation.

One of the requirements presented for concrete beams designed to withstand large plastic deformation is "compression steel should be used . . ." The presence or absence of compression steel would not seem to be critical, depending primarily on the load capacity required. The requirement, also given in the paper, that the beam be underreinforced would appear to be of greater significance in preventing compressive failure of the concrete.

Analyses included in the paper were based on initially peaked triangular loads. This simplified load form has great practical advantages, but its limitations should be kept in mind. The durations considered in the paper are for the initially peaked triangular loadings—it is necessary to know the relationship between this simplified loading and the actual loading if the actual duration is to be determined. Selection of the initially peaked triangular load to represent an actual loading may itself present a difficult task. In the general case one must carry out a response analysis for the actual loading and then select a triangular load that would result in essentially the same response. For the purposes of the paper, however, direct use of initially peaked triangular loads is suitable and clearly demonstrates the influence of load duration.

AUTHORS' CLOSURE

The authors appreciate the comments by Mr. McKee and Mr. Greaves. Mr. McKee has pointed out the desirability of using the resistance-displacement relationship exhibited when beams are subjected to dynamic loads. In Reference 6, this relation is discussed in conjunction with dynamic tests on 6 ft long reinforced concrete beams. It is shown that the resistance-displacement relationship for these beams is in fairly good agreement with the modified static resistance as used in the paper. In general, it is believed that the results obtained from the use of such modified static resistance relations will be sufficiently accurate for design purposes.

Mr. McKee also questions the need for compression steel to enable reinforced concrete beams to withstand large plastic deformation. Reference 11 provides experimental justification for this requirement.

Mr. Greaves comments on the criterion of beam behavior which would allow the beams to deflect to their ultimate deflection. He is concerned with an inexperienced designer incorrectly concluding that a beam which fails has performed better than a beam which suffers no damage. It was not necessarily expected that the first criterion mentioned in the paper would be used as a design criterion—rather it was used as a means of determining the maximum dynamic load which could be applied to a beam. It is expected that a design criterion would be used which expresses the maximum allowable deflection in terms of the span length of the beam. In any case, however,

designers would be expected to use engineering judgment so as to avoid the pitfall posed by Mr. Greaves.

It should be emphasized that the information presented in the paper is in a form adaptable for analysis. In general, however, the engineer will have a greater need for design information. A report is presently in preparation at the U. S. Naval Civil Engineering Laboratory which presents design charts for the selection of beams with suitable sections to resist a given blast loading.

The writers would like to point out that in the preparation of the figures no consideration was given to the increase in concrete strength with strain rate. This would slightly alter the results for small ratios of t_1/T.

REFERENCE

11. Gaston, J. R.; Siess, C. P.; and Newmark, N. M., "An Investigation of the Load-Deformation Characteristics of Reinforced Concrete Beams up to the Point of Failure," *Structural Research Series* No. 40, University of Illinois, Dec. 1952.

Discussion of a paper by Edward A. Abdun-Nur and Joseph J. Waddell:

Control of Concrete Mixes*

By BRYANT MATHER, LEWIS H. TUTHILL, and AUTHORS

By BRYANT MATHER†

The authors' approach to control is generally to be commended, since it is based on sounder scientific principles and at the same time is more practical than many approaches that have previously been employed. The authors speak of "indoctrination . . . in the intent of the specifications" as a "principal accomplishment of the first season's work." They describe the specification requirements and state the criteria for considering that concrete has failed to meet these requirements. The examples of control that are described include some where the concrete produced consistently met the specified requirements, one which started outside the requirements but later came in, and one "that was bad from the beginning." They add that it was gratifying that of some 50 contracts "the end of the first season saw practically all of them meeting the requirements." The authors do not state what penalties were imposed for failure to comply with the requirements or what disposition was made of the concrete that failed to comply. For the reader to visualize the way in which the control was effected it is believed necessary for the picture to be completed by describing the contractual provisions that motivated the contractor to furnish concrete complying with the required control requirements.

The decision to use cements obtained under a specification that limited the alkali content to 0.7 percent, since the alkali-aggregate reaction to be expected was "rather mild" is described as a "calculated risk." It would be helpful in evaluating the circumstances that pertain to this "risk" if additional data could be provided indicating what criteria of mildness might be employed on future work that might justify similar calculated risks.

By LEWIS H. TUTHILL‡

Messrs. Abdun-Nur and Waddell are to be commended on the considerable effort they have made and congratulated for the remarkable results they accomplished on such a widely differing and fast moving operation. The value of the approach they have taken, in the preparation of advanced speci-

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TABLE A—APPROXIMATE STRENGTH VARIATIONS FOR THREE CLASSES OF CONCRETE USED ON NORTHERN ILLINOIS TOLL HIGHWAY

	Class	Cement, sacks per cu yd	Strength, psi
Fig. 2	M	5¾	3150 to 6300
		5¼	3050 to 5450
Fig. 3	K	5½	3000 to 5600
Fig. 4	K	4½	2550 to 5000
Fig. 5	R	7¾	4000 to 7200
Fig. 6	K	5¾	2550 to 4500
		6⅛	2650 to 4500

fications for the work and the on-the-job control they have described, is apparent in the immediate results even as measured by concrete strength tests, and I predict will be further confirmed by an unsurpassed service record in the years to come. It is a procedure and a result well worth emulation on other elements of our great national highway program.

It is most regrettable, in this day of rockets to the moon and submarines traveling under and rising at will through the polar ice, that such a splendid effort to obtain quality and uniformity as the authors have described, has no better means of measurement than the mercurial results of compressive strength tests of concrete test cylinders. It is felt that such varying results leave much to be desired as a means of measuring concrete quality, despite the authors' seemingly casual acceptance of and lack of concern for low tests within 3σ of the average, which are permissible according to the rules of statistical control, so long as the required average strength is obtained. Aside from these rules, it is suggested there is reason to question whether the extremes in strength indicated actually exist in concrete in place.

In view of means used to automatically provide basic uniformity in the concrete mix, such as tested uniform aggregates, automatic weigh batching, moisture meters, finish screening, and W/C, slump, and cement content limitations which were followed, it is seriously doubted that concrete performance at the project actually ranged in approximate strength as given in Table A*.

Nevertheless, such spectacular spreads between highs and lows, and others approaching them, are obtained from test cylinders and make disturbing configurations on control charts such as the authors have presented. But are they real? Is there concrete of this implied range of quality in the project structures and paving, despite all the fine equipment and controls which have been provided? Probably not, unless specification requirements and mix limitations were sometimes flagrantly violated. Most of these requirements, such as high slump and W/C, can be readily noted by inspection, and corrected as they approach limiting values; action on such matters should not and seldom does await the outcome of concrete test cylinders.

*Compiled from data taken from Fig. 2-6.

Moreover, such spreads in strength cannot be explained by any of the discernable or likely variations in slump or W/C. Specifically, it is unlikely that slump ranged more than 2 in. below and 2 in. above the average. Even so, at these extremes, other things being equal, water content would not be decreased more than 7 percent or increased more than 5 percent. The average effect on a W/C of 0.50 would be to increase or decrease it about 0.03 for a range of 0.06. From any of the conventional W/C–strength curves it will be noted readily that strength changes about 100 psi for each change of 0.01 in W/C. Thus from the widest probable range in slump, we can account for only about 600 psi or only about ¼ of the average of the ranges shown above in Table A.

Neither can these ranges be explained by any probable variation in performance of the automatic cement weighing equipment used. Other things being equal, a 6 percent reduction followed by a 6 percent increase from the correct amount would be required to produce the above 0.06 range in W/C, with corresponding results. In a 5¾-sack mix this would imply a variation in measurement of 0.35 of a sack per cu yd or a range of 0.70 of a sack. Accordingly, about four times this variation (1.4 sacks) and range (2.8 sacks) in cement measurement would be required to account for the full average range noted in the strength tests. A range of nearly 3 sacks in the accuracy of cement batching is not credible.

Perhaps statisticians would say that all such things happening at once cause these extremes. It is conceivable, but probably unlikely. For instance, without adjustment of other batch quantities to maintain yield, such extra cement would cause much of the 2-in. reduction in slump, examined above. Consequently, with inspection that would certainly take action on slumps deviating 2 in. from the desired average, by rejecting mixes too wet and adding water to dry ones, and since certainly no samples of these unacceptable mixes would be used to represent job conditions, it therefore seems that the full combined effect of these two variables, which affect strength most, could not occur.

Thus it is difficult to identify and doubtful if there are other factors which could actually change the strength of concrete, as mixed and in place, enough to account for most of the other 2/3 of the spread. It is therefore questionable that such extremes in strength or quality actually exist in concrete in place on such a well controlled job as the authors have described. If they do exist, it is likely that they represent only one batch and not the day's work. The low extreme could just as well have occurred in some untested batch on the day an extremely high value was obtained, yet neither would represent the general character of concrete placed that day or either day.

It is not easy to get a high test from low strength concrete but there are many things that can reduce a test result from what it should be. Omitting all aspects of dishonesty, high strengths are more believable than low ones. Improper high strengths have been obtained by such means as taking the sample before adding water to truck-mixed concrete, adding extra cement to

the test batch, selecting a very low-slump batch for the test batch, drying the cylinders before testing, or high rates of loading during testing.

But there are many things which can affect a test result unfavorably, the prominent among these are: unrepresentative sampling; irregular or rounded ends on test cylinders as molded; poor consolidation; uneven distribution of aggregate; inadequate early protection from drying, temperature extremes, and physical damage; improper capping; eccentric loading; and sluggish indicators on testing machines. Any of these careless practices can cause results that are misleading and adjustments based on these results often are not justified judging by the actual quality of concrete produced.

Before any penalty or expensive action is taken because of such low strength test cylinders, a thorough review should be made of test procedures since they may have contributed to the low results. Another precaution should be a careful study of cement composition and cube strength results to determine if the low concrete strengths correlate with low cement tests or with lower than usual amounts of C_3A or C_3S. If there has been such a change in composition, 90-day strengths will usually be up to expected strength.

If there is still doubt as to the strength and quality of concrete in place, the concrete test hammer may be used to make comparative observations. To do this, several score of readings are taken on sound, well-cured, surfaces of the concrete in question and compared with corresponding readings on concrete surfaces that are considered satisfactory. After corrections have been made for any difference in age of the two surfaces, it will be reasonably apparent whether the questioned concrete should be further investigated. Any further doubt may be resolved by comparing the strength of drilled core specimens from the same two concretes.

Some may question use of the concrete test hammer on the grounds that the variations and extremes in its readings usually exceed considerably the questioned variations and extremes among the compressive strength test results. But it is not the purpose of this discussion to question the validity of these compressive strength test extremes. It is rather to question what they are too commonly taken to represent; i.e., *all* the concrete of the day, or shift, or area from which the test cylinders were taken. It is true that test hammer readings vary widely due to the blow occasionally hitting a stone at the surface, an air pocket just under the surface, or other local variables of small area and negligible significance. But if an ample number of readings are taken and are well distributed over the concrete in question, and if low readings prove to be well distributed, the average of all the readings will be a fair indication of the quality of this concrete, despite the range in individual readings. Actually, because the high and low readings are not representative of the general quality of concrete because of the local conditions mentioned above, the official test hammer instructions recommend that "the best 10 out of 15 readings be used." By best they mean the ten closest to the mean value. The reasons for the extremes in compressive strengths of test cylinders are much more obscure, and thus such extremes should be considered only statis-

tically; it is what they truly represent that is important. When they are believed to represent the day's work, this conclusion may be seriously questioned; further data is needed to prove that all concrete placed that day is of this quality.

Concrete test cylinders are a convenient and relatively inexpensive means of obtaining an indication of the strength of job concrete. In general, they are a reasonably reliable means of establishing a certain measure of average concrete quality. Nevertheless, their limitations should not be forgotten, and they should not be used as the only and final basis of judgment where expensive acceptances, replacements, or cement additions are at stake. As is so well pointed out in "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)," ACI JOURNAL, V. 29, No. 1, July 1957 (*Proceedings* V. 54), p. 10: "inflexible strength requirements are unrealistic and control of the pattern of results rather than individual values is the most appropriate basis for both specifications and the general assessment of results."

AUTHORS' CLOSURE

The authors appreciate the thoughtful discussions by Messrs. Mather and Tuthill and will attempt in the following paragraphs to answer some of the questions.

Mr. Mather's question regarding "contractual provisions that motivated the contractor to furnish concrete complying with the required control requirements" is answered in principle by the last paragraph on p. 948. In practice, the specifications required a lower average strength for the better uniformity, other things being equal, and a higher average strength for lesser uniformity. This in turn meant that less cement was needed to obtain the lower average which accompanied high uniformity, thus higher profit to the contractor; and more cement to meet the higher average strength required by lower uniformity, thus raising the contractor's cost. This is the incentive that motivated the contractor to work toward a higher degree of uniformity.

As for penalties for failure to comply with the concrete strength requirement for any given operation, this is always a matter of engineering judgment and a decision for the engineer administering the contract to make when faced with a specific instance and its existing factors. This type of control being a new approach unfamiliar to contractors and inspectors alike, it was expected that there would be a short "starting adjustment period" in which both the contractors and inspectors would be getting their bearings, and during which period some strengths would be somewhat lower than the specification requirements. In these starting periods, placements were limited to small volumes in relatively noncritical locations, simply because the work had not gained the momentum that developed into large daily yardages later on.

The policy adopted was to carry "training" to the field under actual operating conditions, and to work with the contractor and inspectors to adjust and modify operations to permit meeting specification requirements. As long as the contractor was cooperative, and the strength reasonably close to the requirement, nothing was done except to help him smooth his operation, but in other cases, the cement content was raised, thus forcing the contractor to do something about his work in self-defense against the higher cement content.

The limitation of 0.7 percent for the alkali in cements to be used with certain potentially reactive aggregates, was a matter of specification interpretation rather than a requirement. The specifications did not contain a limit on alkalies in cement, but had an acceptance provision based on satisfactory service records in similar structures in the area. This provision was invoked whenever a high-alkali cement was proposed for use with aggregate that showed potential reactivity. There being no time available for expansion bar tests, reliance was placed on petrographic examinations and judgment. When such conditions occurred, structures in which such combinations had been used were examined visually in the field and small samples were recovered and examined petrographically for signs of reaction. In most instances it was concluded that a rather mild reaction existed, which although probably not detrimental *per se*, might open minute cracks to the freezing action of water, which would then do the major damage. Where such questionable combinations occurred, most cement suppliers were able to furnish cement much lower in alkali than the limit of 0.7 percent which was judged adequate. In one instance, where this was not feasible, switching the cement to other contracts with nonreactive aggregate was found possible.

In other words, the answer to Mr. Mather's questions is that the contractual meeting of the specifications is more a matter of engineering judgment and interpretation than of mathematical statistics, as in the field many factors are intermixed and cannot be isolated in the same manner as in a research project in the laboratory, and at the same time, decisions have to be made without the benefit of answers from thorough research studies. Statistical control does, however, provide a mathematical tool that will permit the engineer to ascertain, within given probabilities, whether the concrete delivered to the forms is produced under "control" conditions or whether there are variations that have assignable causes that can be corrected. Of course, the quality of the concrete shown by the control cylinders and that in the structure are not necessarily the same, as the cylinders and the structure part company soon after they are made. This is the major point raised by Mr. Tuthill.

Mr. Tuthill questions the validity of the use of concrete compressive strength cylinders as a measure of the variations in the strength of the concrete in place in a structure. The authors would like to point out that the paper deals with the control of concrete mixes, and that nowhere in the paper

have they attempted to translate that into the control of variables in placing, curing, and protecting the concrete, and its final quality in the structure.

After concrete control cylinders are made, they are subject to some uncontrollable variations, which were maintained at a minimum on this project through measures of uniformity described in the paper. The test results represent the potential qualities of the concrete as delivered to the forms, as shown by the compressive strength tests made and handled under standard conditions, which are the best that our present knowledge of the subject has to offer. After the concrete is deposited in the forms, it too, is subjected to a set of variables that cannot be controlled even with the best procedures—variables of placing, vibrating, segregating, consolidating, finishing, curing, protecting, exposure to early mechanical and temperature variations, and many other unknowns. These latter variables are not subject to measurement and can be minimized only through mechanization, automation, and inspection. Whether these variables have the same net effect on the finished structure as the variables to which the cylinders are subjected is a matter of conjecture.

Proof on this point is lacking and inadequate. Some studies have been made comparing control cylinders with cores taken from the structure, but it must be noted that the cores, after they are taken, are subject to the same variables as the cylinders. Such things as conditions of moisture at testing, capping procedures, capping materials, accuracy and adjustment condition of the testing machines, and many other factors affect the tests in both cores and cylinders. In addition, due to the later age at which cores are taken, they would normally show a higher strength than 28-day cylinder strengths. A review of a large amount of published and unpublished data reveals that there are many cores that show lower strength than corresponding cylinders, while a large number show higher strengths. Another factor that makes comparisons uncertain, is that the published material comes more frequently from large well controlled projects than from small uncontrolled structures. One accessible source of such data is Table 3 in the Bureau of Reclamation manual,[1] in which both higher and lower strength cores are compared to cylinder strengths.

It is the opinion of the authors that using best practices of placing, curing, and protecting concrete a structure is probably better (strength-wise) than the cylinders representing the concrete that went into the forms. This applies mostly to structures placed under proper control and inspection. On the other hand, structures placed sloppily without specifications and inspection, under highly competitive workmanship and bid prices, particularly on such projects as houses, driveways, sidewalks, small buildings, etc., the sum total of which represents a larger volume of concrete than the larger more adequately controlled projects, will probably have more variations and be of lower strength than the cylinders supposedly representing them.

Mr. Tuthill doubts that the structures contain concrete of the range shown by the cylinders, and gives a detailed analysis to show that the extremes

TABLE B—ANALYSES OF USBR JOBS

Year	Coefficients of variation, percent
1949	15.4
1950	14.3
1951	14.3
1952	14.1
1953	13.8
1954	14.5

shown in the control charts could not have existed, in such a well controlled job. It is the opinion of the authors that such variations in concrete *do* exist in well controlled jobs, and that such variations are far worse in poorly controlled or noncontrolled jobs than those shown in the paper. Perhaps the best study on variations in concrete control is the series of yearly analyses made of Bureau of Reclamation jobs a few years ago. Table B gives weighted average coefficients of variation.

The coefficients of variation shown in the paper range from 10.7 to 16.2 percent, with an average comparable to that experienced by the Bureau of Reclamation. Therefore, the extreme variations shown in the paper are apparently similar to those experienced on other well controlled projects. The reasons Mr. Tuthill was unable to justify such extremes from his calculations are:

1. Cement strength variations were not considered.
2. The factors used in the calculations were taken from average curves (instead of the extreme variation of those average curves), and are then compared with the extremes of the control curves presented.

The variability in the strength of cement has been frequently discussed in the literature. Among these discussions are the recent paper by Walker and Bloem[2] and the paper by Wing and Ruettgers[3] in 1943, in which the latter conclude that 40 percent of the cylinder strength variation was ascribed to cement variations.

A study by Wing, Price, and Douglass[4] reported in 1944 indicates that cylinders made and tested in both research and field laboratories gave an average within-batch coefficient of variation of 3.4 percent, indicating that the compressive cylinder test *per se* is reasonably reliable, whereas under the same conditions and in the same laboratories, the average batch to batch coefficient of variation was found to be 12.8 percent or over three times the within-batch value. If such differences can exist under laboratory conditions, it is the opinion of the authors that the coefficients of variation shown in this paper represent excellent control when it is realized that under field conditions it is impossible to attain the uniformity in batching which is attained in the laboratory.

That there were occasional batches that were too high in slump, or that were not mixed properly, or in which there were errors in weighing or in air entrainment, the authors have no doubt, but that such were general practices, the authors are sure was not the case. On this project, if the testing program called for a test, and the batch happened to be an off-batch, the test was made anyway, as should be the case in all random sampling for statistical control. Not to sample under such conditions would be a serious error, and would provide a record that *does not* represent the facts. For

statistical control to function properly and for the data from it to be valid, sampling must be frequent enough so that the "law of large numbers" can be applied, and it must be random. One sample to represent a day's production would not, on the average job, qualify under the former, and selective sampling in which the inspector deliberately chooses the batches from which cylinders are made would void the "random" sampling requirement.

It must be stressed that on a fast moving paving or structural job, by the time an inspector makes a test to objectively determine the slump, there would have been several batches dumped on top of the one tested and mixed together so that it would be impossible to reject the poor batch. Once a "controlled" operation is reached, it is much better to accept the statistical fact that random variations are going to occur and not lose sleep over the individual variations, as long as the over-all average is adequate and no extremes occur outside of the $\pm 3\sigma$ range. In the long run, control under such a scheme is far superior to standing at the forms watching every single batch that goes in and visually rejecting every one that appears too wet.

At no time on this project did one single test represent a day's work, and it is inconceivable that anyone would consider one test per day as a useful control on a large project. In fact, such an approach would not be statistically valid. The specifications required that three sets of cylinders be made for each 2000 sq yd of pavement or fraction thereof placed in any one day, and similarly, three sets for each 100 cu yd or fraction thereof for structural concrete placed in one day. An average day's paving on one contract would have required nine sets of cylinders, and many structures the same number.

Unless an operation has a badly skewed distribution curve, a high strength test is as likely to occur for a given coefficient of variation as a low one. Actually, unadulterated dishonesty on the part of inspectors, who have been put on the spot of enforcing unenforceable minimum strength specifications, has been observed in the making of cylinders prior to the addition of tempering water, or in over rodding each layer in the making of the cylinders. Under such circumstances, a good cylinder strength is shown in the record, but does not represent the concrete in the forms. One of the advantages of working with averages, and not being disturbed by individual variations, is that it permits the inspector to be honest without making him feel that he is going to be disgraced or lose his job if a cylinder turns out low in strength.

Regarding the various items that could contribute to lowering of cylinder strengths, statistical treatment of data recognizes the fact that some of these variables together with variables tending to increase the strength of cylinders do occur in various combinations each time a cylinder is made, and allows for such random and uncontrollable variations as being the order of the day for a given operation. As long as actions taken to correct mixes are clearly designated in the specifications and the basis of adjustment clearly given as being based on cylinder strength, and the contractor enters into a contract based on these, it does not matter what the corresponding condition in the structure might be, as long as it meets *its* specification requirements.

By this, it is meant that if the compressive strength to be met is clearly outlined in the specifications as being the result of cylinders made, cured, and tested under certain conditions, the cylinder strength is what should determine compliance with the contract, and not the probability that the strength of the concrete in the structure might be higher. The authors do not see any hardship on the contractor in being asked to fulfill his contract—he entered into it with his eyes open and could read the requirements clearly.

Paving work and much of the structural work go too fast to practically utilize Mr. Tuthill's suggested analysis of cube strengths, C_3A and C_3S content of the cement. It must again be stressed, that once the operation is under satisfactory control, one has reasonable assurance that the cylinder strengths will fall in the proper pattern when the 28 days come around, and they then become something for the record—to confirm that the job was done properly. Controlling the operation in such a manner as to get the proper results at 28 days is the key to the situation and tests the skill and knowledge of the concrete engineer.

As for the test hammer, many investigations, among them the thorough study by the Bureau of Public Roads,[5] have shown that the readings are sensitive to many variables (much more so than the compressive strength cylinder test), and have concluded that such readings are a good guide to judgment, when used judiciously, but cannot replace compressive strength cylinders for control purposes. Many years ago, the Bureau of Reclamation used a test based on the same principle, utilizing a steel ball dropped from a standardized height, and after much data had been collected, it was found to be unreliable, and its use has been discontinued. This type of test depends on the modulus of elasticity more than on the strength, and the rebound tests one small spot only, and each spot tested is different from the others; whereas to measure the composite strength of concrete, a summation or an integration of the effects of all the components is needed—a function that the compressive strength cylinder does reasonably well.

If the pulse velocity method could be developed and simplified to where it can be correlated with strength, it would be an ideal method of measuring the strength of concrete in place, as it measures a summation or integrating of the various components of the concrete.

The closing quotation in Mr. Tuthill's discussion is exactly the basis on which the specifications of the Northern Illinois Toll Highway were developed. The authors are happy to see that in spite of Mr. Tuthill's apparent disagreement in the body of the discussion, he finishes by agreeing with the basic principles the authors used on this project.

The authors' viewpoints may be summarized as follows:

1. The contractual meeting of specifications and their enforcement is more a matter of engineering judgment and interpretation than of mathematical statistics.
2. This paper deals with control of concrete mixes, and not with the strength of concrete in structures. The compressive strength test is still, to the best of the authors' knowledge, the best available tool for this purpose.

3. For valid statistical conclusions, the number of batches sampled must be *sufficiently large* to make the application of the law of large numbers valid, and the sampling must be *random*, otherwise statistical analysis will give an erroneous picture.

4. The variations shown in the paper are not "spectacular," but represent rather good control judged by present day concrete know-how.

5. The judicious use of the test hammer, although a good tool to guide judgment, involves many more variables than the compressive test cylinder, provides spot readings rather than an integration of the component action of concrete, and is therefore in the opinion of the authors, useless as a tool for control purposes.

6. It is hoped that before long, developments in pulse velocity techniques will result in simplified methods that will permit, with proper calibrations, the nondestructive measurement of concrete strength in given structures. With the increased use of the equipment it is probable that the cost of the apparatus will be reduced to bring it within the reach of the average job.

The authors are grateful to Messrs. Mather and Tuthill for their discussions which have afforded the opportunity to clarify some points that may not have been sufficiently clear in the original paper.

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Discussion of a paper by A. M. Neville:

Role of Cement in the Creep of Mortar*

By A. DE SOUSA COUTINHO, KIYOSHI OKADA and AUTHOR

By A. DE SOUSA COUTINHO†

One of the most important properties of concrete which is still hidden under a thick veil of mystery is creep. Actually there is as yet no theory which satisfactorily explains this phenomenon; any explanation of it one tries to give clashes against some other experimental data with which it does not agree.

At the present state of our knowledge, we think that the most promising way for studying creep in concrete is to search for a relationship between creep and other already known properties of concrete such as strength, which is a more or less known and mastered property.

For this reason, we consider the author's work to be one of the most important carried out in the field of creep up to the present. We arrived at similar conclusions 5 years ago in a work which, being written in Portuguese, did not come to the author's attention. In that paper[37] we studied the cracking of cement, mortar, and concrete due to their restrained shrinkage.

This study on cracking was conducted by using concrete rings which were cast around metallic rings having low deformability but sufficient to allow the measurement of stresses in them. The stresses in the metallic ring being known, it is then easy to calculate the stresses in the concrete ring. The concrete ring had an 8 cm square cross section with a 65 cm interior diameter.

The paramount factors studied were those influencing cracking and stresses developed in restrained concrete shrinkage. These factors were richness of cement, maximum size of aggregate, quantity of mixing water, and cross section of the specimen. The main conclusion reached in this work was that the cracking tendency was greater when the strength of concrete was lower. Stresses due to restrained shrinkage change little, only a few kg per sq cm, when each of the above factors varies. But the variation of tensile rupture strength is much more pronounced, and, when concrete strength is low, cracking occurs more frequently.

The study also included creep tests which, for the sake of convenience, were made on compression specimens. These creep tests were performed on specimens 7 days old, cured at 70 percent relative humidity and at a tem-

*ACI JOURNAL, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55), p. 963. Disc. 55-62 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

†Research Engineer, Head of Concrete and Cementitious Materials Division, Laboratório Nacional de Engenharia Civil, Lisbon, Portugal.

perature of 20 C. The specimens tested were prisms having square cross sections with sides ranging from 8 to 20 cm long. Applied stress for the creep test was 30 kg per sq cm in every case.

In these creep tests, different values for richness of cement, maximum size of aggregate, quantity of mixing water, and cross section were used.

The instantaneous elastic strain (after several cycles of loading and unloading at 30 kg per sq cm) was measured at the beginning and the end of the test. The ultimate strength of the specimen was also measured at the end of the 1½-year test.

For each loaded specimen there was a companion specimen for correction of shrinkage. The ultimate strengths of these specimens were determined at the same time as the loaded specimen. The experimental data showed that loaded and unloaded specimens had exactly the same ultimate strength. The experimental data also showed this important fact: the greater the ultimate strength, the less the creep.

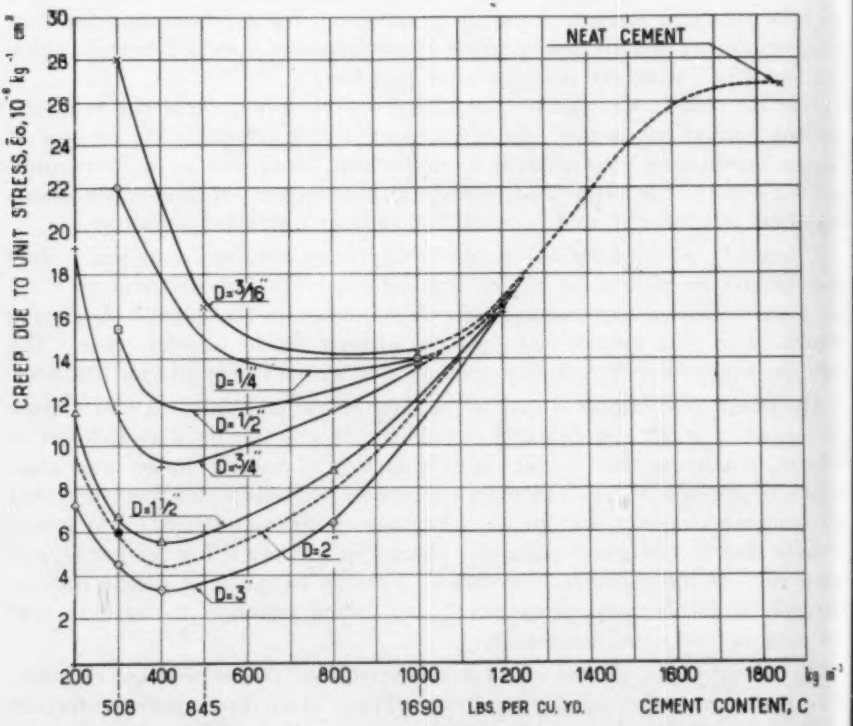

Fig. A—Effect of cement content and maximum size of aggregate on the compressive creep of concrete. Dashed lines concern concretes with 2 in. maximum size aggregate, and concretes with a cement content greater than 1000 kg per cu m (1690 lb per cu yd), experimental results lacking. Creep was determined at 1 year. Section of specimens: 20 x 20 cm (8 x 8 in.)

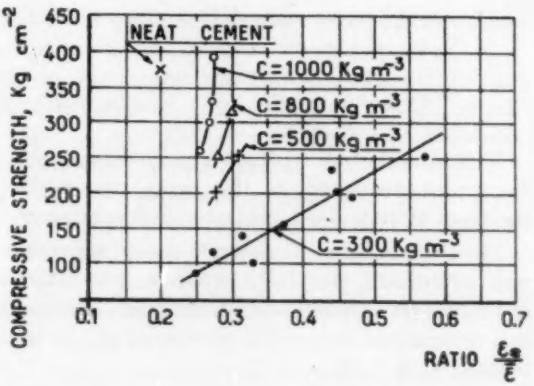

Fig. B—Relation between compressive strength, σ_r, creep strain, $\bar{\epsilon}$, and instantaneous elastic strain, ϵ_e. Results obtained were from compressive creep tests

Some of the important results of creep tests concerning the richness of cement and maximum size of aggregate are shown in Fig. A. This figure is also an answer to the "question" asked by the author, p. 982: "It is not possible however, to state whether richness of mix is a factor in creep. This can be established only from tests with a graded aggregate on a wider range of mix proportions."

Actually Fig. A shows that the richness of mix is a factor in creep and that it has a double effect on it. In the lean concrete with a maximum size of aggregate greater than, say, ½ in., when the cement content increased up to 400 to 500 kg per cu m, creep decreases as strength increases. The same is true for concrete with a maximum size of aggregate less than ½ in., for a cement content below 1000 kg per cu m. When the cement content exceeds these limits both the ultimate strength and the creep increase, which is due to overabundance of cement paste in the concrete.

From these tests, a relationship is obtained between the ratio $\epsilon_e/\bar{\epsilon}$ (ϵ_e = mean of elastic strains measured at the beginning and at the end of the test; $\bar{\epsilon}$ = creep strain) and the rupture stress of concrete, σ_r. This relationship is shown in Fig. B.

The general equation of these curves is

$$\sigma_r = \sum_{o}^{i} a_i \left(\frac{\epsilon_e}{\bar{\epsilon}}\right)^i$$

In a first approximation we can write

$$\sigma_r = a_1 + a_2 \frac{\epsilon_e}{\bar{\epsilon}}$$

and, introducing the secant modulus of elasticity

$$E = \frac{\sigma}{\epsilon_e}$$

where σ is the stress under which creep takes place,

$$\bar{\epsilon} = \frac{a_2 \sigma}{E (\sigma_r - a_1)}$$

This formula well represents the results obtained by the author for mortars. An expression similar to this one is also derived from the data obtained in the tests of restrained concrete shrinkage rings.

Tensile strength of concrete rings subjected to restrained shrinkage, σ_r, was determined; the elastic strain, ϵ_e, was determined by means of the stresses in restrained concrete shrinkage rings, $\bar{\sigma}$, and the modulus of elasticity, E, was determined on control specimens of the same concrete subjected to pure tension ($\epsilon_e = \bar{\sigma}/E$).

Creep strain, $\bar{\epsilon}$, was determined from the difference between the strain the specimen would have if free (shrinkage, δ) and the formerly calculated actual strain ($\bar{\epsilon} = \delta - \bar{\sigma}/E$). Plotting $\epsilon_c/\bar{\epsilon}$ against σ_r, a linear relationship is obtained (Fig. C), which is well represented by Eq. (1).

What is important to remember is that this conclusion was reached from relaxation tests (restrained shrinkage is practically a relaxation test).

A remarkable conclusion attained in our work was that the laws governing relaxation are identical to those governing creep; that is, concrete relaxation is greater the lower the strength, and the law of inverse proportionality between relaxation and rupture strength also holds good.

Parallelism between the conclusions reached in both works, ours and that under discussion is, then, obvious. But, in our work we introduce the modulus of elasticity, a factor neglected by the author. Eq. (1) as well as the general idea of the relationship between creep and strength are only approximate, it is quite possible that we are both right because perhaps factor E is not of paramount importance.

In our paper, the cracking of cement, measured both in neat cement paste and mortar, was also studied. In these tests, rings of the above materials were cast around metallic rings, with a square cross section 2.5 cm on a side,

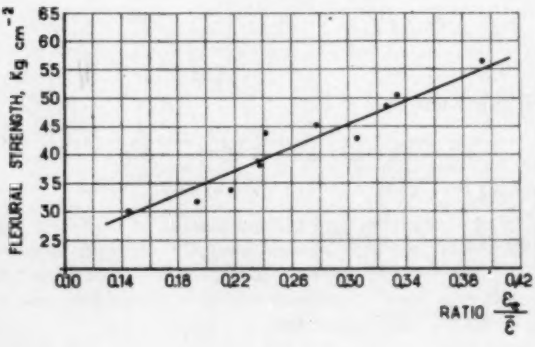

Fig. C—Relation between bending strength, σ_r, of restrained concrete shrinkage, creep strain, $\bar{\epsilon}$, to which concrete would be submitted if it were free under the stress $E\,\delta$, and instantaneous elastic strain, ϵ_e, due to the stress $\bar{\sigma}$. Cement content: 300 and 500 kg per cu m. Water content and maximum size of aggregate were variable

and 20 cm interior diameter. But the studied cements all had similar properties, and it was not possible, for this reason, to draw any conclusions.

In the meantime as the work on this subject was carried on for some time, we were able to obtain some different cements, thus enlarging the field of our observations, and last year a paper on the "Influence of the Type of Cement on Its Cracking Tendency" was presented at the symposium on the influence of time upon strength and deformation of concrete, held at Munich by RILEM, Nov. 17-20, 1958. The results obtained in this paper for the case of concrete are given in the following generalization: creep of cement varies as the inverse ratio of its rupture strength.

In this case, creep of cement was determined in a 1:1 mortar and the rupture strength in a totally different 1:3 mortar.

This fact is illustrated in Fig. D. In this diagram, there is no need to consider the modulus of elasticity:

$$\sigma_r = \frac{K_1}{\epsilon_o + K_2} + K_3$$

where σ_r is the rupture strength of the mortar (that is to say, of the cement) and ϵ_o is the specific creep of another mortar made of the same cement (that is, the creep of the cement).

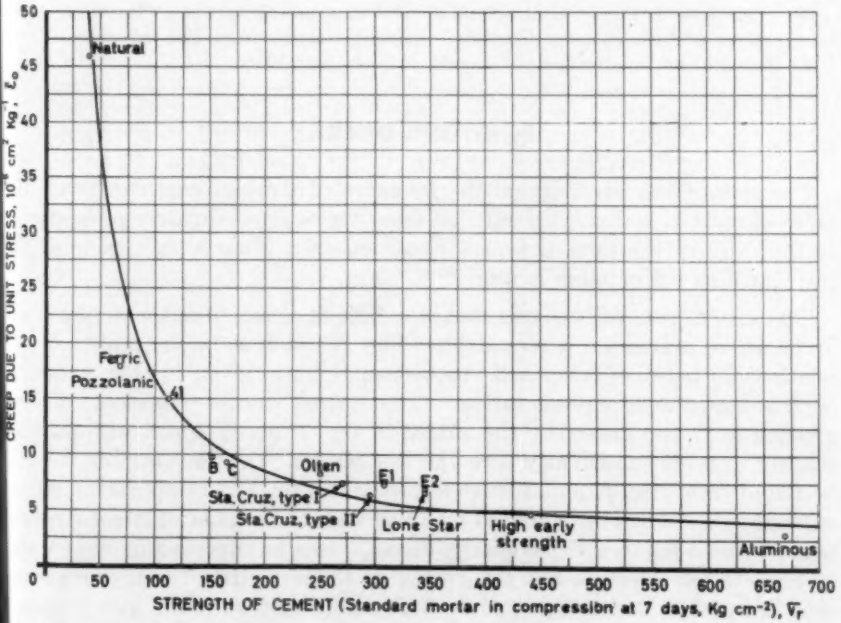

Fig. D.—Creep of cement at 60 days measured in a 1:1 mortar, as a function of its strength measured in a 1:3 standard test

The influence of the relative humidity of the air was not considered in our tests on creep and relaxation. The author deserves praise for his consideration and demonstration that *the influence of this factor on creep is exclusively due to the influence of this factor on the rupture strength of the concrete.* It is then clearly demonstrated that creep varies as the inverse ratio of the strength. This variation is due to one of the following factors:

(a)—Maximum size of the aggregate
(b)—Water content
(c)—Cement content
(d)—Cement quality
(e)—Curing
(f)—Plasticizing agent[38]

One of the most troublesome properties of creep remains to be the influence of the specimen cross section. While the rupture strength decreases when the cross section increases, creep decreases also. Why? This is a question to which so far no adequate answer is available.

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By KIYOSHI OKADA*

The author has investigated the influence of stress-strength ratio on the creep of mortar, and pointed out that creep seems approximately proportional to the ratio at the time of load application, even when a high value of the ratio such as 0.8 or more is used.

The writer, however, believes that it is difficult to say from the present data if the above conclusion is reasonable. Fig. E and F show the replots of the relationship between creep and stress-strength ratio given by Fig. 1 and 2 in semilogarithmic representation. The dotted curves represent the regression lines also drawn by the author in Fig. 1 and 2, which will not necessarily coincide consistently with the test results. On the contrary it could be found from Fig. E and F that logarithmic values of creep versus stress-strength ratios could be expressed by a line folded at a point of stress-strength ratio of about 0.3 to 0.4. Similarly, Shank's data as cited in Fig. 6, can also be replotted as shown in Fig. G. In Fig. G Richart's data[39] concerning short time creep tests under high values of stress-strength ratio are also schematically replotted.

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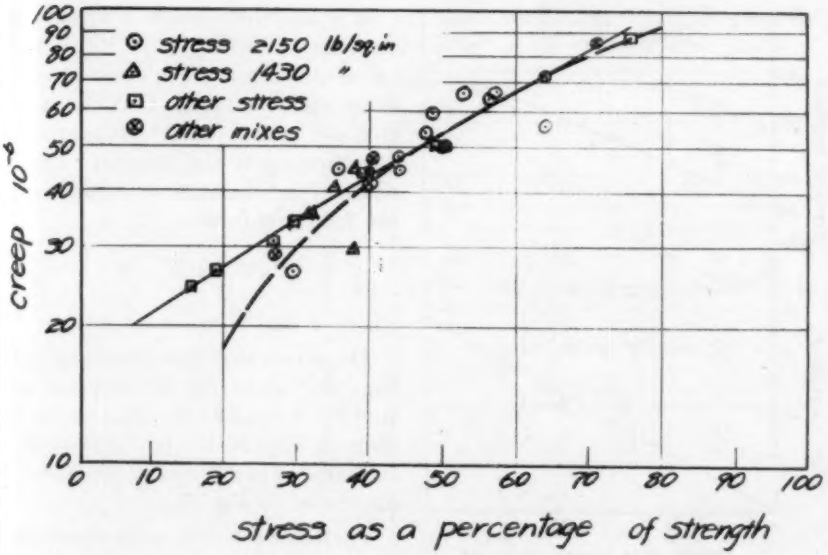

Fig. E—Creep of mortar stored under humid conditions for 60 days

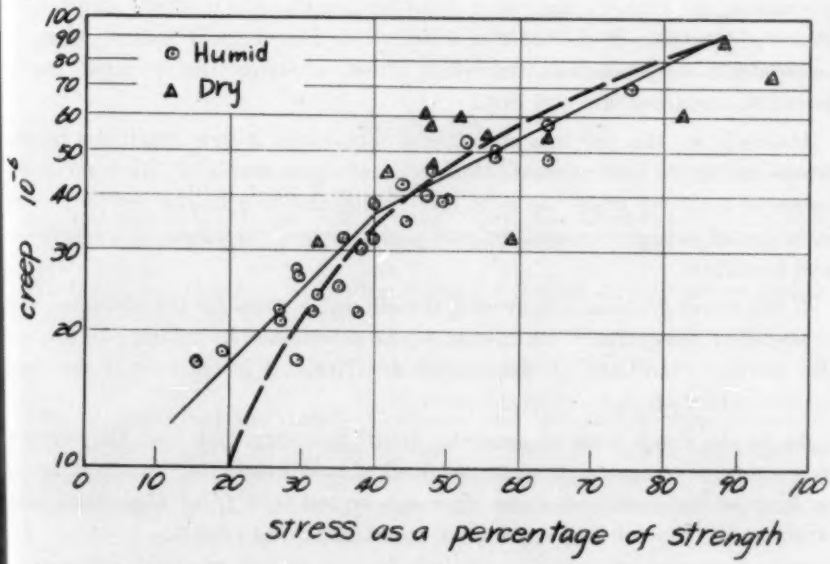

Fig. F—Creep of mortar stored under load for 20 days

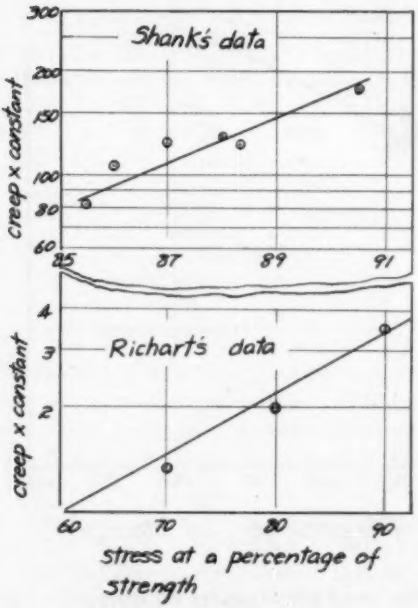

Fig. G—Creep versus stress–strength ratio (Shank's and Richart's data)

It seems there exists a linear relationship between the logarithmic value of creep and the stress-strength ratio, and this means that the stress-strength ratio has a fundamental influence upon the creep ϵ_c after a definite load duration as expressed by the following form:

$$\epsilon_c = A\, e^{\sigma/f_c'} + B$$

where A and B are constants.

The above equation coincides with Eq. (9b) given by Freudenthal and Roll.[40] Actually the data given by them in Fig. 10-12 (pp. 1131-1135 in the same paper[40]) can be replotted as illustrated in Fig. H.

Therefore it seems quite reasonable that the conclusion as stated by the author holds good so long as the stress-strength ratio is fairly low, but may not when the ratio is as high as 0.8 or more. Allowing for the above discussion, the author's suggestion is interesting that the ultimate compressive strain of mortar is of the order of 0.002. But does it really indicate that the same strain corresponds to the failing stress, whenever the specimen is subjected to sustained stress or not?

If this is so, the problem of extreme importance is how much the mortar creeps under its environment; that is, how much does the initial strain increase or multiply when the total creep is applied to a mortar specimen that is subjected to a given sustained load under certain conditions of temperature and humidity.

If the above problem is answered, the allowable stress for the sustained load or so-called "creep limit" for mortar can be determined by taking into account the storage conditions of the specimen. Nothing instructive is discussed by the author on this important point.

As to the creep limit of concrete, Ban[41] has suggested that the concrete may carry a compressive sustained load almost indefinitely without failing so long as the sustained stress does not exceed 0.76 f_c' or the initial total strain at the time of load application is not larger than 0.001.

The writer believes that more tests are needed to clarify the creep limit of mortar or concrete.

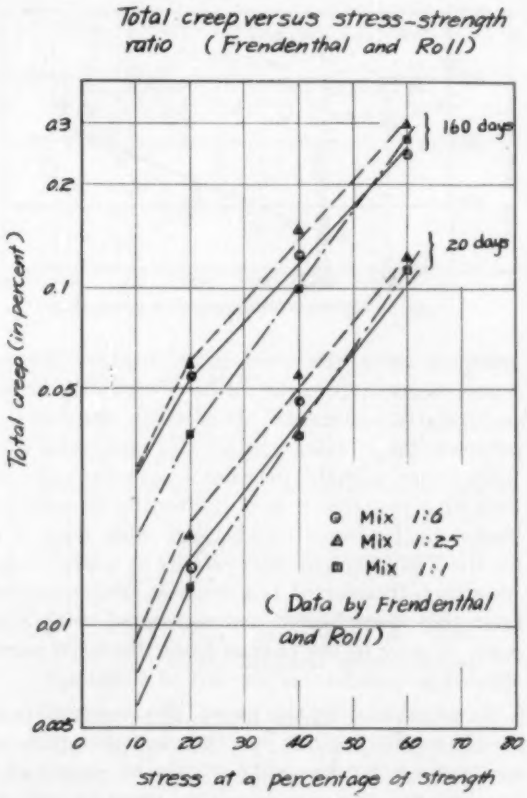

Fig. H—Total creep versus stress–strength ratio (Freudenthal and Roll)

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AUTHOR'S CLOSURE

The contributions of Messrs. Coutinho and Okada are most interesting and it is particularly gratifying to hear of work on creep—an important though sometimes a little neglected property of concrete—in various parts of the world; the ACI JOURNAL seems an admirable medium for such an international exchange of information.

Mr. Coutinho's tests confirm the relation between the stress-strength ratio and creep, suggested in the author's paper. Since these tests, dealing

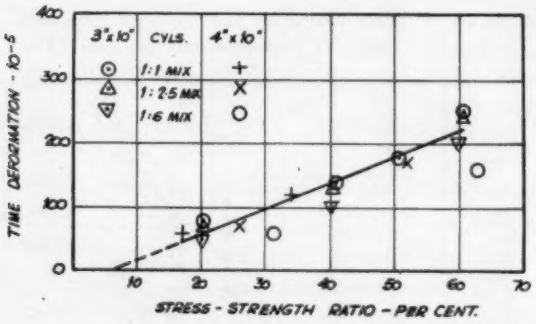

Fig. I—Time deformation against stress-strength ratio for Freudenthal's[40] concrete after 40 days under load

primarily with the cracking of cement, mortar, and concrete, differed in many respects from the author's tests, the close agreement of the findings is particularly significant. It is worth observing, too, that the general relation between the stress-strength ratio and creep is also applicable when the mix proportions and the maximum aggregate size are varied.

In this connection it is interesting to refer to Freudenthal's tests,[40] mentioned by Professor Okada. In these tests the creep specimens were cured for the first fortnight after casting at a relative humidity of 100 percent. They were then transferred to a room at 60 percent relative humidity. A fortnight later they were loaded and continued being stored at a humidity of 60 percent. Owing to the change from 100 to 60 percent the companion specimens showed a considerable amount of shrinkage.

As mentioned in the paper, the assumption that shrinkage and creep are simply additive is not justified, and that part of shrinkage which occurs at a *constant* relative humidity cannot be separated from creep. If then Freudenthal's[40] data are recalculated so as to include shrinkage in the total time deformation and plotted against the stress-strength ratio the curves of Fig. I, J, and K are obtained. These tests are particularly revealing as they cover a wide range of mix proportions, the aggregate-cement ratio varying between 6 and 1, and yet, all the mixes together plot as recognizable straight lines. The only discrepancy is apparent in the case of 4 x 10-in. cylinders of the 6:1 mix which show a consistently low creep at all ages.

It is not possible, however, to say with certainty whether these lines are truly straight or, as observed by Professor Okada, there may be a discontinuity between two lines at a stress-strength ratio of about 30 to 40 percent. The probability of the existence of two distinct creep-stress relationships was mentioned at the top of p. 979 of the original paper. However, the difference between the semilogarithmic curves of Professor Okada (Fig. E and F) and the author's regression lines is not believed to be of great importance, at any rate for creep after long periods under load.

In this connection it may be noted that the curves based on Freudenthal's test results for periods under load of 40, 100, and 160 days (Fig. I, J, and K respectively) show that the greater the age the lower the value of strength

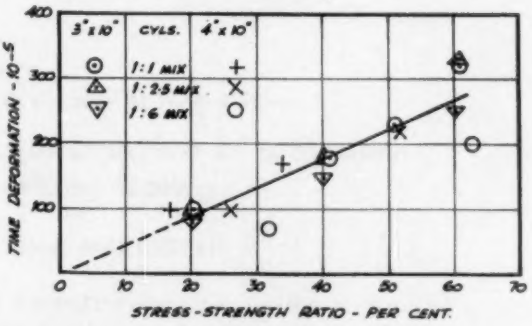

Fig. J—Time deformation against stress-strength ratio for Freudenthal's[40] concrete after 100 days under load

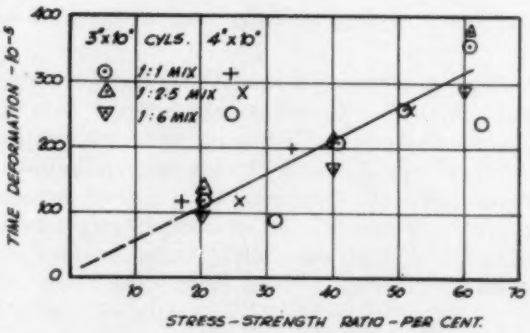

Fig. K—Time deformation against stress-strength ratio for Freudenthal's[40] concrete after 160 days under load

for which, from the relationship postulated, no creep would be expected. The same tendency was apparent in the author's tests. A possible explanation is that creep catches up with shrinkage only after a time. A further contribution on this topic is presented by the author in the RILEM *Bulletin*, 1959.

Discussion of a paper by Walter E. Riley:

Design and Construction of a Modern Parking Garage*

By J. MAKARETZ†

The author presented an interesting approach to the problem of garage construction. The writer would like to add some information concerning similar structures and express his point of view as to the general design approach.

In 1956 a six-story garage was designed by Thomas Worcester Co. in Boston, under the writer's supervision, for the city of Providence, R. I. The design consisted of a 14-in. uniform flat plate supported on circular columns. Span lengths varied, but the maximum spans were 34 ft with 12-ft cantilevers. Shear heads such as those indicated by Mr. Riley were used. Due to a larger dead load the shear stresses were greater than in the Phoenix garage, but even for smaller spans two concentrical shear heads were used, and for the 34-ft spans, three concentrical shear heads were used. The concrete shear stresses used were below the allowable value.

In accordance with a paper by E. M. Rensaa (ACI JOURNAL, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), pp. 695-715), a reduction of shear stresses is justified in continuous beam designs. It is believed that this is also true for flat plates with considerable lateral rigidity and high negative moments, as in this case.

It appears to me that the author's design for shear reinforcement was in accordance with a brochure for reinforced concrete put out by Portland Cement Association in 1948 entitled "Shearhead Reinforcement for Flat Plate Floors." The author probably used a 90 psi shear value in accordance with ACI 318-56. Because shear problems are not entirely understood, it is believed that in using the full shearing capacity of concrete in such structures as described above, there would result an unsafe condition.

The author also mentioned that during the placement of concrete at high temperatures (110 F) that the floor camber was provided for before the concrete set. It seems rather difficult to place and finish about 40 cu yd of concrete in one floor panel (34 ft x 31 ft 3 in.), and then introduce a 1-in. camber between initial and final set without using any retarder (the author does not mention any) and without affecting the concrete quality.

*ACI JOURNAL, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55), p. 985. Disc. 55-63 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 31, No. 3, Sept. 1959, Part 2 (*Proceedings* V. 55).

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The second similar structure (flat plate with drop panels), is supported by 22-in. cylindrical columns 25 ft on centers with 10 ft 6 in. cantilevers. The design was by Badger Manufacturing Co., under the supervision of this writer, for Sinclair Oil Refining Co., Pasadena, Tex.

The exchanger structure is a two-story, 40 ft high, 160 ft long building, designed as 12-in. slab, with 20 in., 8 ft square, drop panels at the columns. The combined live and dead load amounts to 450 psf. To accommodate high shear stresses, specially fabricated bar trusses were used (four in each direction for every column head). Further details can be found in an article by the writer in *Petroleum Refiner*, scheduled for printing in the fall, 1959.